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SETTLEMENT OF OIL STORAGE TANKS

Andrew M. Braswell, Jr.,[1] A. M. ASCE
(Proc. Paper 1863)

SYNOPSIS

This paper presents the results of observations covering a period of six years on the actual settlement of steel tanks that are used for the storage of petroleum products. The survey included tanks of various heights and diameters situated on sites with widely varying soil conditions within the top few feet. A comparison is also made between the actual and theoretical settlement of a large tank for typical soil conditions prevailing in the area covered.

INTRODUCTION

The region covered by this study is located on the coastal plain bordering the Gulf of Mexico near the upper end of Galveston Bay. The geological formation consists, in general, of a layer of clay roughly 300 ft. thick overlying a series of sand strata of various thicknesses. The clay formation is interspersed with numerous and irregular sand pockets and seams. A generalized north-south profile through the area has been plotted from the logs of water wells and is shown in Fig. 1. A 200' x 48' tank filled with water would exert a pressure of approximately 465 psf at the lower boundary of the clay. (Throughout this paper, when a tank size is given the first number denotes the diameter and the latter number denotes the height.)

It is desirable to predict the probable settlement of storage tanks for many reasons. The design and flexibility of connecting appurtenances, grade, drainage layout, and inter-connection of piping between tanks are all dependent to some extent upon the settlement that will be experienced. The original objective of the study was to determine the amount of settlement that existing tanks had undergone. Accordingly, elevations were taken on the

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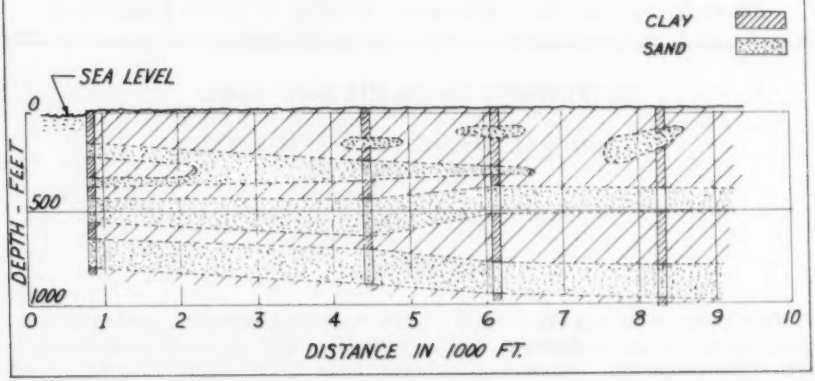
1. Sr. Design Engr. Humble Oil & Refining Co., Baytown, Tex.

FIGURE 1

NORTH-SOUTH PROFILE
THROUGH PLANT

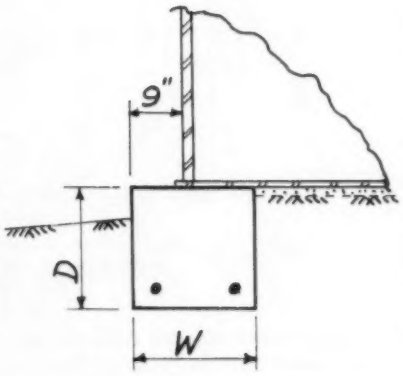


quarter-points of forty-nine tanks ranging in size from 30' x 30' to 145' x 48'. Many of these tanks had been in service 20 to 30 years. When the recent elevations indicated that some of the tanks had risen anywhere from 3 inches to 5 feet, it was concluded that little reliable settlement data could be obtained from this source. Unfortunately, the area covered by this study is within a region where movement of benchmarks is not uncommon. In addition, it is quite probable that the original elevation of many foundations did not correspond to that shown on the construction drawings. It is worth noting, however, that the survey of the 49 old tanks indicated that the differential settlement between any two points on the ring of a particular tank rarely exceeded one and one half inches. The tanks were supported on continuous concrete rings as shown in Fig. 2A. In some instances, the foundation ring had failed from lateral pressure, in which case the differential settlement amounted to a foot or more.

It was obvious at this stage of the survey that the design of the standard foundation rings should be re-examined. The basis for the design of the new rings is illustrated in Fig. 2C. In general, a layer of black organic clay, or gumbo, extends to a depth of 8 ft. with a much stiffer tan or yellow clay below this depth. The depth of the ring "D" is set so that the passive resistance of the soil outside the ring will withstand the lateral pressure exerted on plane O-a between the bottom of the ring and the top of the yellow clay. The width of the ring "W" is set so that the shell and liquid load on the ring produces a unit soil pressure beneath the foundation approximately equal to the unit pressure beneath the interior of the tank when the tank is full of water. Sufficient longitudinal reinforcing bars are provided in the ring to resist the hoop stress caused by lateral soil pressure, as well as the bending moment caused by eccentric loading on the ring. Typical ring dimensions for several size tanks are shown in Fig. 2B. Thus far, the design has proved to be entirely satisfactory in preventing shear type failures in tank foundations.

FIGURE 2-A

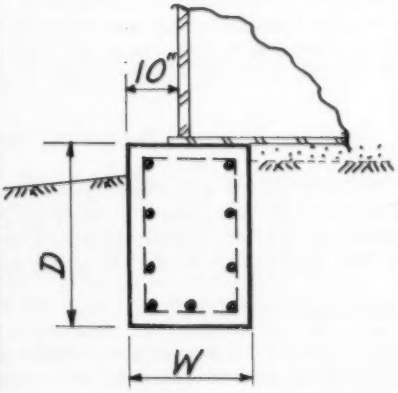
TANK DIA.	D	W
60 FT. OR LESS	18"	18"
60 - 120 FT.	24"	24"



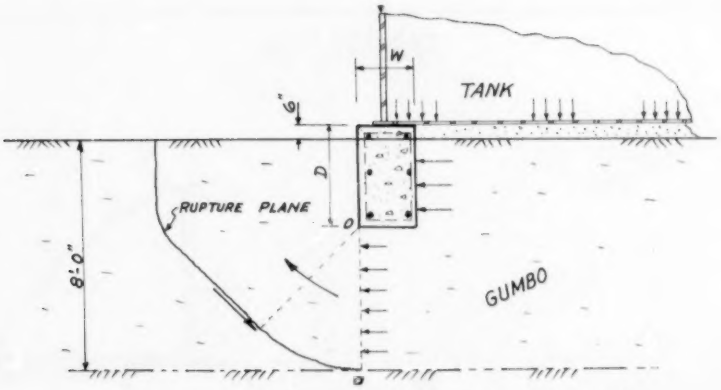
TYPE "A" FOUNDATION RING

FIGURE 2-B

TANK DIAMETER		HEIGHT 30	40	48
30	D	18"	24"	36"
	W	18"	18"	18"
60	D	18"	24"	36"
	W	18"	18"	18"
80	D	24"	24"	36"
	W	24"	24"	24"
100	D	24"	24"	36"
	W	24"	24"	24"
120	D	24"	24"	36"
	W	24"	24"	24"
150	D	24"	24"	36"
	W	24"	24"	24"



TYPE "B" FOUNDATION RING



DESIGN BASIS - STD. FDN. RING
FIG. 2-C

It was further concluded from the preliminary survey that reliable settlement data could only be obtained from settlement levels taken on new tanks subsequent to their construction. The records of 10 such tanks are given in Table I. Several of the tanks were built in pairs and offer an excellent opportunity to compare the relative settlement of tanks constructed on natural soil versus those constructed on a fill. In comparing the settlement of different tanks, however, it should be noted that the projection of the foundation ring outside the shell was changed during the course of this study. This projection was originally 4" rather than 10" as shown in Fig. 2B, which resulted in higher soil pressures under the foundation. For this reason, the soil pressure beneath the foundation has been given in Table I for the various conditions of loading of each tank. All tanks included in this paper have a projection of 4" except Tanks 3, 4, and 6, which have a projection of 10".

Results of Observations

The construction site and settlement record of 8 of the 10 tanks included in this paper are described in detail below. The site for the remaining two, Tanks 8 and 9, is essentially identical to the site for Tank No. 3, so settlement data only are given for these two tanks.

Tanks 1 and 2

These two 140' x 48' cone roof tanks were constructed simultaneously in 1953 at 350 ft. on centers. Tank No. 1 was placed on natural soil with the bottom of the foundation 3 ft. below grade. Tank No. 2 straddled a 9 ft. deep slough as shown in Fig. 3. Eight core borings and numerous laboratory tests were made of the subsoil at the site. Core Borings 1 and 2 are representative of the soil conditions at Tanks 1 and 2 respectively. All soft material was excavated from the slough beneath Tank No. 2 and a rolled clay fill was placed, the thickness varying from zero to 12 ft. below the bottom of the

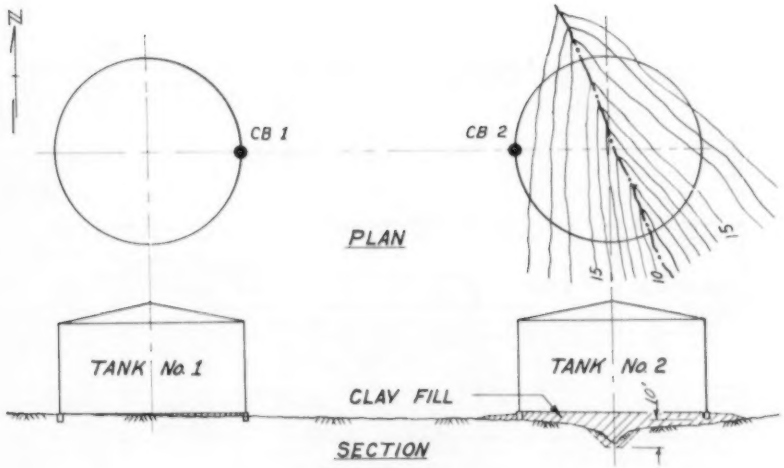


FIGURE 3

foundation ring. Optimum dry weight of the fill material was 110 lbs./cu. ft., and compaction to 98% of optimum was obtained by means of sheeps-foot rollers. The average depth of oil maintained in these tanks is about 25 ft. Settlement has been as follows:

Tanks	During Water Test		Total to Date	
	Maximum	Differential	Maximum	Differential
1	2-1/2"	1-1/4"	3-7/8"	1-1/4"
2	1-5/8"	1/2"	4"	1-1/8"

Laboratory tests indicate that the total settlement above represents approximately 60% of the ultimate under the imposed loading.

Tanks 3 and 4

This pair of 150' x 48' floating roof tanks was constructed simultaneously in 1954 at 375 ft. centers. Tank No. 3 was built on natural soil while Tank No. 4 straddled a firewall, with one quarter of the ring located in a slush area associated with an old dismantled tank. Soil conditions at the site are shown by Core Boring No. 3. In an effort to limit differential settlement of the circumferential foundation ring of Tank No. 4, a 7 ft. deep trench was excavated completely around the periphery of the tank as shown in Fig. 4. The trench was then backfilled to the bottom of the foundation with a select clay which was compacted by means of sheeps-foot rollers to 95% of the Proctor density.

Tanks 8 and 9 are two 150' x 48' tanks located at a site for which the soil conditions are very similar to those existing at Tank No. 3, and as shown in Core Boring No. 3. Both tanks were built on natural soil. An average depth of 25 ft. of oil is maintained in these four tanks.

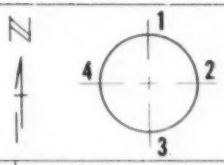
TABLE I

OIL STORAGE TANK SETTLEMENT DATA

TANK NO.	SIZE	DATE	ELEVATIONS 1	2	3	4
1	140 X 48	8-13-53	21.00	21.03	21.00	21.00
		12-30-53	26.02	26.06	25.98	25.96
		1-11-54	25.89	25.94	25.87	25.75
		3-19-58	25.70	25.87	25.76	25.64
2	140 X 48	11-19-53	20.96	21.02	20.95	20.95
		11-19-53	24.89	25.27	24.88	24.88
		11-23-53	24.82	25.22	24.81	24.80
		11-27-53	24.77	25.18	24.77	24.75
		12-10-53	24.83	25.23	24.82	24.80
		3-19-58	24.62	25.03	24.61	24.55
3	150 X 48	7-7-54	28.48	28.46	28.51	28.50
		11-1-54	28.15	28.14	28.25	28.29
		3-19-58	27.77	28.08	27.90	27.85
4	150 X 48	7-9-54	28.49	28.51	28.51	28.50
		11-4-54	28.50	28.50	28.47	28.45
		11-10-54	28.37	28.34	28.30	28.33
		3-19-58	28.27	28.17	28.12	28.23
5	100 X 40	1-7-54	29.34	29.35	29.34	29.33
		1-11-54	29.25	29.26	29.24	29.25
		5-3-55	29.13	29.15	29.16	29.15
		3-19-58	28.96	29.01	29.01	28.99

Table I
Oil Storage Tanks
Settlement Data

TANKS
DATA



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SETTLEMENT DATA (FT.) MAX.	SETTLEMENT DATA (FT.) DIFF.	PRESSURE UNDER RING #/FT²	REMARKS
		525	TOP OF RING - TANK NOT CONST.
		1130	MARK ON SHELL - 5′ ± ABOVE RING
			TANK EMPTY
.21	.10	3510	MARK - TANK FULL OF WATER
.32	.13	1630	12′ OIL IN TANK
		1130	TOP OF RING - TANK EMPTY
		1130	LUG ON SHELL - TANK EMPTY
.08	.03	2320	LUG ON SHELL - ½ FULL WATER
.13	.04	3510	LUG ON SHELL - FULL OF WATER
.08	.04	1130	LUG ON SHELL - TANK EMPTY
.33	.09	1660	12′-9″ OIL IN TANK
		450	TOP RING - TANK NOT ERECTED
.33	.12	3350	TANK FULL OF WATER
.71	.33	2460	37′ OIL IN TANK
		450	TOP RING - TANK NOT ERECTED
.07	.06	1755	14′ WATER IN TANK
.21	.09	3350	TANK FULL OF WATER
.39	.17	2500	38′ OIL IN TANK
		930	TOP RING - TANK EMPTY
.10	.02	3300	TANK FULL OF WATER
.21	.03	2030	TANK CONTAINS 18′ OIL AT 270°F
.38	.05	2130	CONTAINS 21′ OIL AT 270°F

TABLE

OIL STORAGE

SETTLEMENT

TANK NO.	SIZE	ELEVATIONS				
		DATE	1	2	3	4
6	160 X 48	11-12-56	25.61	25.58	25.58	25.58
		4-26-57	25.59	25.58	25.57	25.56
		6-20-57	25.42	25.44	25.45	25.43
		1-17-58	25.37	25.41	25.43	25.39
7	73'-4" X 40'	8-24-54	17.00	16.98	16.99	16.99
		10-1-54	16.97	16.93	16.94	16.96
		10-13-54	16.75	16.70	16.75	16.79
		10-15-54	16.69	16.60	16.78	16.73
		10-18-54	16.64	16.57	16.62	16.70
		10-19-54	16.62	16.52	16.61	16.68
		10-20-54	16.61	16.50	16.59	16.67
		10-22-54	16.61	16.49	16.60	16.68
		10-29-54	16.64	16.53	16.63	16.71
		3-28-55	16.60	16.53	16.63	16.68
		3-11-58	16.38	16.36	16.56	16.57
8	150 X 48	5-7-54	29.32	29.30	29.31	29.33
		5-18-54	29.03	29.06	29.00	29.01
		5-24-54	29.12	29.13	29.07	29.08
		3-19-58	28.75	28.77	28.51	28.43
9	150 X 48	5-7-54	29.09	29.09	29.08	29.04
		5-24-54	28.85	28.80	28.85	28.81
		3-19-58	28.63	28.47	28.53	28.63
10	100 X 40	12-30-53	29.29	29.30	29.29	29.28
		5-3-55	29.09	29.09	29.04	29.06
		3-19-58	28.94	28.94	28.87	28.89

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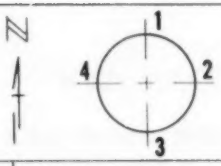
TANKS
DATA



SETTLEMENT DATA (FT.)		PRESSURE UNDER RING #/FT^2	REMARKS
MAX.	DIFF.		
0		525	TOP RING - TANK NOT ERECTED
.02	.02	1250	TOP RING - TANK EMPTY
.19	.06	2900	TANK FULL WATER
.24	.09	1900	AFTER 7 MO. SERV. - $\frac{1}{2}$ FULL OIL
		100	TOP RING - TANK NOT ERECTED
.05	.02	885	TANK EMPTY
.28	.08	2200	TANK FULL OF WATER
.38	.17	2200	TANK FULL OF WATER
.41	.12	2200	TANK FULL OF WATER
.46	.15	2200	TANK FULL OF WATER
.48	.16	2200	TANK FULL OF WATER
.49	.18	2200	TANK FULL OF WATER
.45	.17	885	TANK EMPTY
.45	.14	1000	CONTAINS 17 FT. OF OIL
.62	.20	1000	
		1050	TOP RING - TANK EMPTY
.32	.08	3440	TANK FULL OF WATER
.25	.08	1050	TANK EMPTY
.90	.37	2920	36.5 FT. OIL IN TANK
		1050	TOP RING - TANK EMPTY
.29	.06	3440	TANK FULL OF WATER
.62	.21	3050	40 FT. OIL IN TANK
		930	TOP RING - TANK EMPTY
.25	.04	2030	TANK CONTAINS 18' OIL AT 270^{0}F
.42	.07	2130	TANK CONTAINS 21' OIL AT 270^{0}F

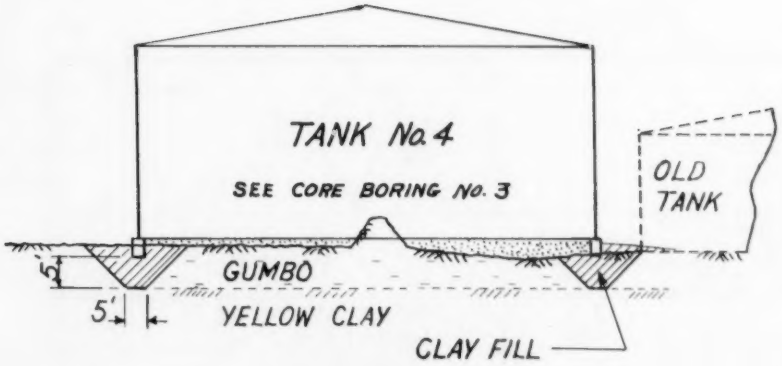


FIGURE 4

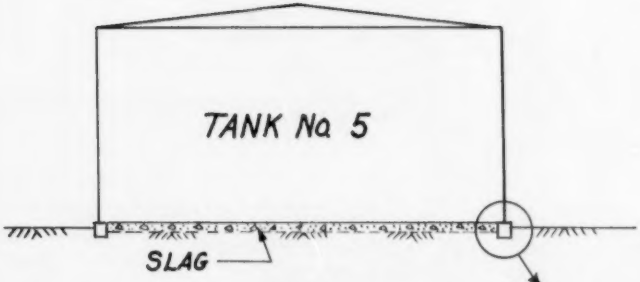


FIGURE 5A

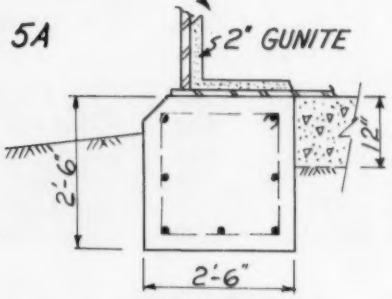


FIGURE 5B

Settlement has been as follows:

Tanks	During Water Test		Total to Date	
	Maximum	Differential	Maximum	Differential
3	4"	1-1/2"	8-1/2"	4"
4	2-1/2"	1"	4-3/4"	2"
8	4"	1"	9-1/2"	4-1/2"
9	3-1/2"	3/4"	7-1/2"	2-1/2"

Tanks 5 and 10

These two 100' x 40' tanks were constructed simultaneously in 1954 at 250 ft. on centers. They are unique as compared to the other tanks included in this survey because the oil stored is maintained at a temperature of 270° F. The tanks are completely lined on the inside with 2" of shotcrete for insulation and have a 12" layer of slag within the ring, as shown in Fig. 5A. The detail of the foundation ring is shown in Fig. 5B. The soil at this site is very similar to that at Tanks 3 and 4 and as shown in Core Boring No. 3. An average depth of 18 ft. of oil is maintained in these tanks.

Settlement of these tanks has been as follows:

Tanks	During Water Test		Total to Date	
	Maximum	Differential	Maximum	Differential
5	2-1/2"	1/4"	4-1/2"	1/2"
10	3"	1/2"	5"	3/4"

The settlement for Tanks 3, 4, 5, 8, 9, and 10 represents more than 90 per cent of the ultimate settlement that can be expected under the imposed loads.

Tank No. 6

This 160' x 48' floating roof tank was constructed in 1957, and like Tank No. 2, straddled an old creek about 12 ft. deep. The site had previously been used as an incinerator area, the refuse from which had been spread over the creek bed with a bull-dozer. The entire area had been filled and leveled a few years later with a sandy clay material that was merely dumped and spread with no effort towards compaction. Core Boring No. 4 describes the soil conditions and Fig. 6 shows a cross-section through the site. By the time construction began on Tank No. 6 the sandy clay fill had reached a well compacted state by natural consolidation. The incinerator material, however, contained at least one of practically every item known to man, including 5 gallon drums and 55 gallon barrels. All of the incinerator refuse within an area extending 10 ft. outside the periphery of the tank was excavated and backfilled to proper grade with a select clay, compacted under the constant supervision of a soils consultant. Settlement of this tank during the water test was 2" with a differential settlement of 3/4". The total settlement after 7 months service was 2-1/2" with a differential settlement of 3/4". On the average, a depth of oil equal to one half the height is maintained in this tank.

Tank No. 7

This 73' x 40' tank was constructed in 1954 for the storage of petrochemical products. Considerations other than structural dictated that, if at all feasible, the tank should be located on a site that had been used a few years

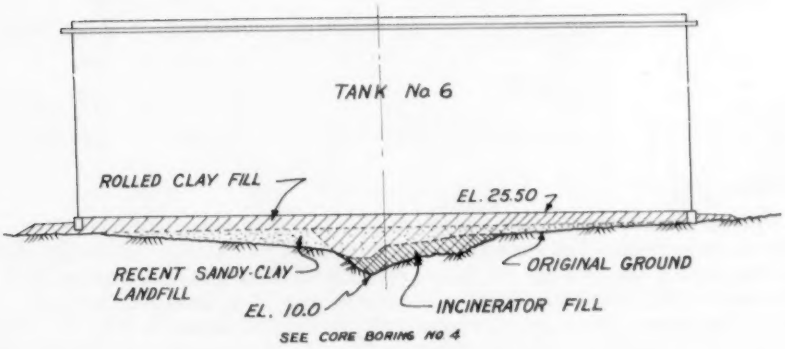


FIGURE 6

earlier as a disposal area for dredgings. The 7 ft. of hydraulic fill had subsequently been covered with 2 ft. of land fill. This was the situation at the site when the tank was constructed, as shown in Fig. 7A. The sludge and sand represents the initial dredgings and was of about the consistency of creamy style peanut butter. The top layer of clay and gumbo was fairly compact. Consideration was given to driving wood piles under the tank with a heavy concrete cap, and as an alternate to driving continuous steel sheet piling around the periphery of the tank. In view of the extensive area in which the soil conditions were as represented by Fig. 7A, considerable savings could be realized in the construction of future tanks in this area if such expensive measures were not necessary. The shearing resistance of the dredged material was estimated to be 225 psf, based upon the height to which open cuts would stand. It appeared that there was an even chance that the soft material would not flow appreciably under the load to be imposed. It was decided, therefore, to take a calculated risk and use only a concrete ring. It was realized that jacking up and releveling the tank might be required periodically. The top of the concrete ring was set 2'-6" above grade to allow for considerable settlement. The tank was filled with water and allowed to stand for 9 days to see if any displacement of the soft sludge would occur. Settlement amounted to 6 inches and was surprizingly uniform. Differential settlement of the ring was only 2". Total settlement to date has been 7-1/2" with a differential settlement of 2-1/2". The pattern of settlement during the water test is shown in Fig. 7B. The first 12 days represents time spent filling the tank; emptying occurred from the 21st to the 28th day.

Comparison of Theoretical and Actual Settlements

The total settlement that a tank undergoes is composed of (1) that due to consolidation of the soil over a long period of time, and (2) an "elastic" type deformation due to the maximum short time loading that occurs during the water test. Total settlement due to consolidation of the soil has been computed for a 150' x 48' tank based upon a long time load of 24 ft. of oil having a

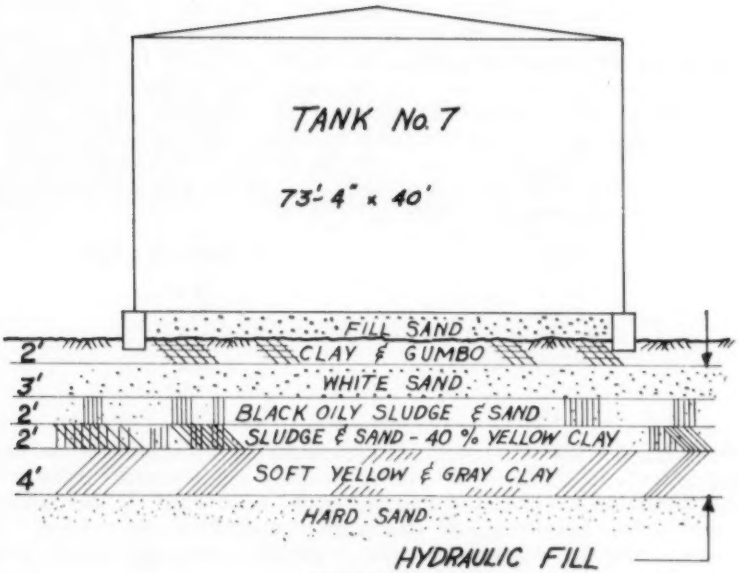


FIGURE 7A

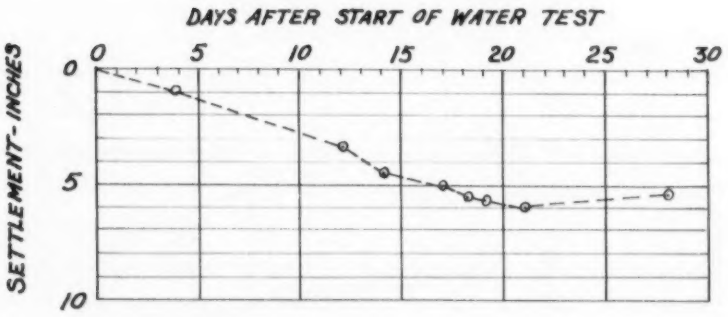


FIGURE 7B

specific gravity of 0.85, and having a foundation ring as shown in Fig. 2B. The computation was based upon the soil conditions illustrated in Core Boring No. 5, which is typical for the greater part of the area involved in this study. The round black dots in the "symbol" column denote depths at which samples were taken for laboratory consolidation tests. The e-log P curves from these tests are shown in Fig. 8. Except for the top 8 ft. layer of gumbo, it was assumed in the settlement calculations that the laboratory e-log P curves represented the recompression curve of a preloaded clay. Changes in void ratio with pressure were thus read directly from the laboratory curves. Compression of the top layer was computed as virgin consolidation due to the difference in the imposed pressure over the pre-consolidation pressure. The calculated settlement at the edge of the tank was 6.9", with the various subsurface strata contributing as follows:

Depth	Consolidation of Layer
0- 8'	2.9"
8-18'	1.1"
18-32'	1.3"
32-48'	0.8"
48-68'	0.5"
76-104'	0.3"

The "elastic" type settlement that occurs during the water test has been computed for a 100 ft. diameter tank by means of the Newmark[2] influence charts, assuming E = 1,000,000 lbs. per sq. ft., and u = 0.5. It was further assumed that the foundation ring was so proportioned that the unit soil pressures under the ring and the center of the tank were approximately equal.

2. "Influence Charts for Computation of Vertical Displacements in Elastic Foundations', by Nathan M. Newmark, published by the University of Illinois press, 1947.

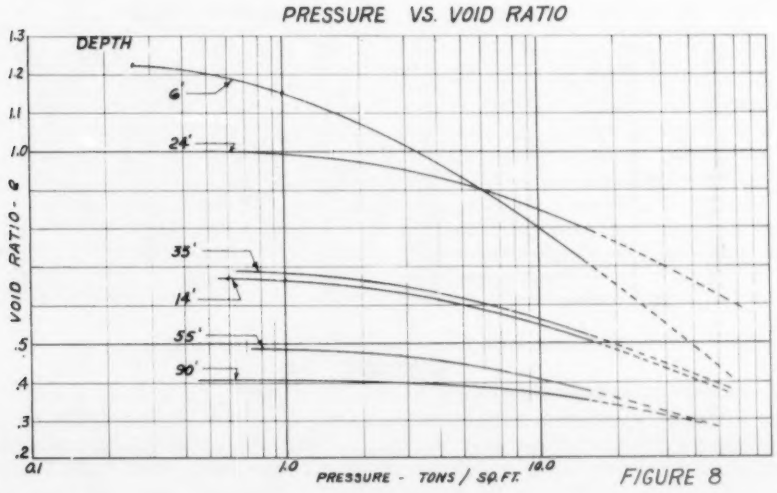


FIGURE 8

The settlement for a tank of any other diameter "D" is equal to the ratio D/100 times the value shown for a 100 ft. tank. Observed settlements at the edge of actual tanks have been converted to the equivalent settlement of a 100 ft. tank by means of the above ratio and are plotted in Fig. 9. The plotted settlements are the maximum that occurred anywhere on the ring.

Combining the two components of settlement for a 150' x 48' tank gives a total settlement at the edge of 9.6 inches. This compares with an average observed settlement of 8.0 inches.

In closing, the effect of differential settlement upon the gaging accuracy of a tank might be discussed briefly. Assuming that the nature of the settlement is such that the tank tilts and the bottom remains a plane surface, Fig. 10 shows the gaging error. It is interesting to note that, within practical limits, the gaging error per foot of gage is independent of the tank diameter and depends only upon the magnitude of the differential settlement.

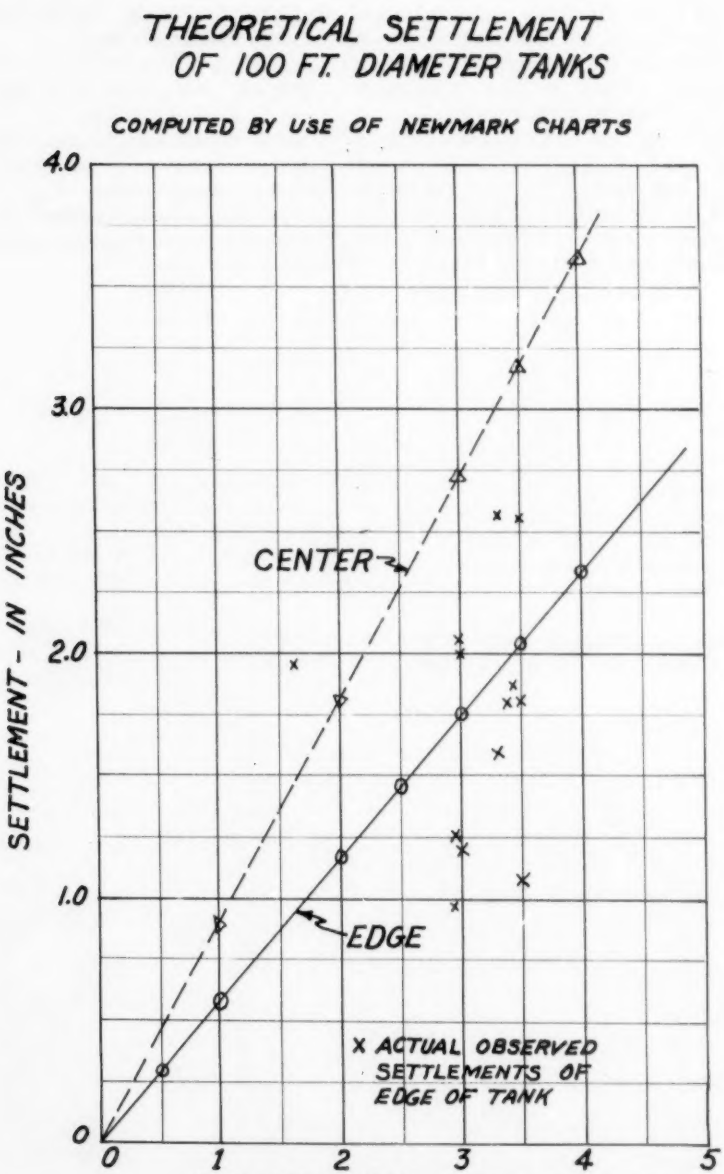

THEORETICAL SETTLEMENT
OF 100 FT. DIAMETER TANKS
COMPUTED BY USE OF NEWMARK CHARTS
SETTLEMENT - IN INCHES
4.0
3.0
2.0
1.0
0
CENTER
EDGE
X ACTUAL OBSERVED
SETTLEMENTS OF
EDGE OF TANK
0
1
2
3
4
5
SOIL PRESSURE - KIPS PER SQ. FT.

FIGURE 9

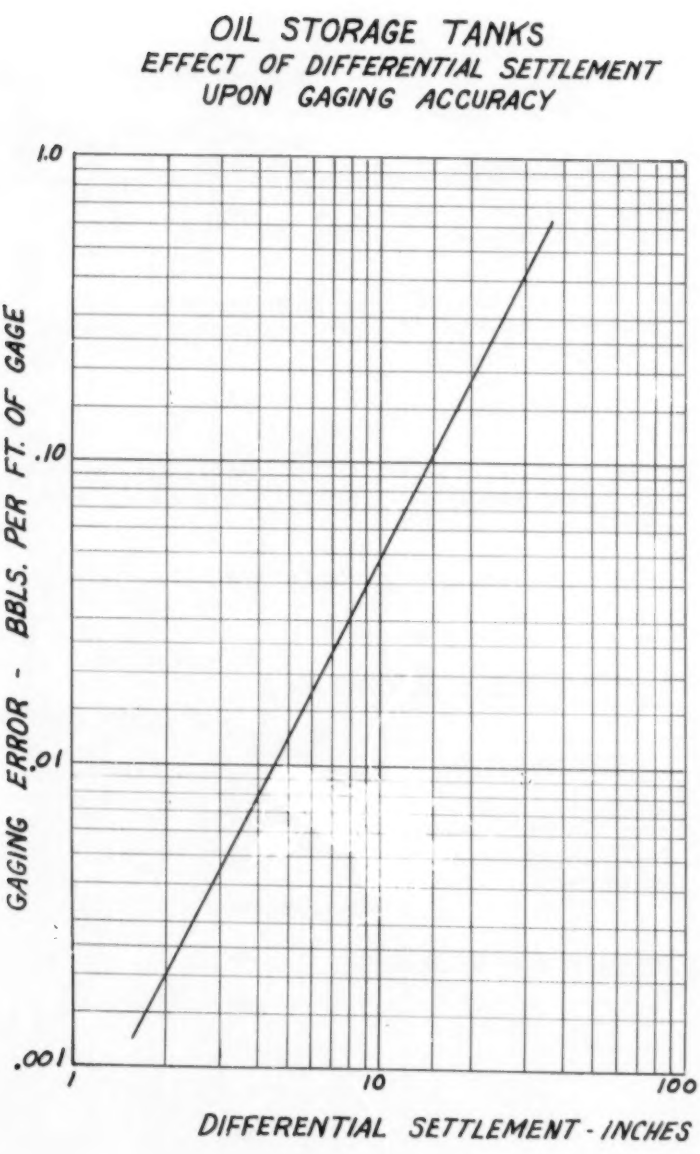
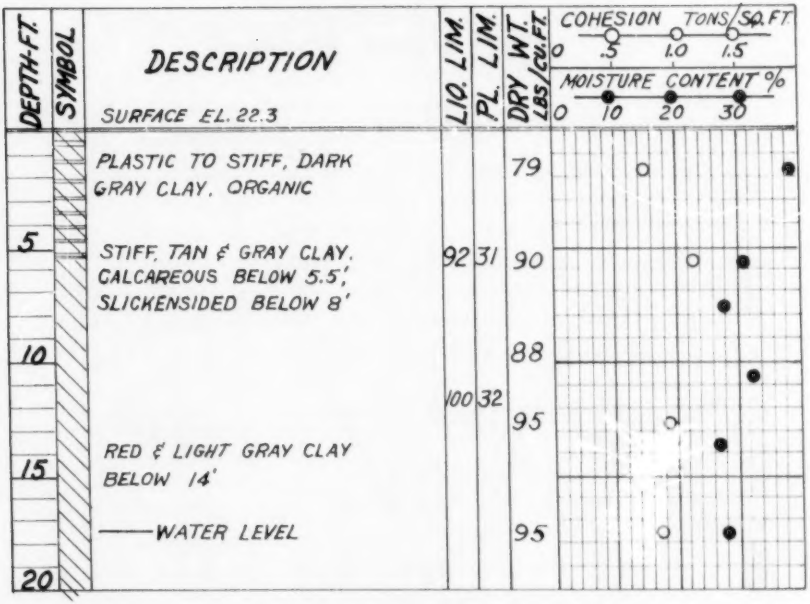
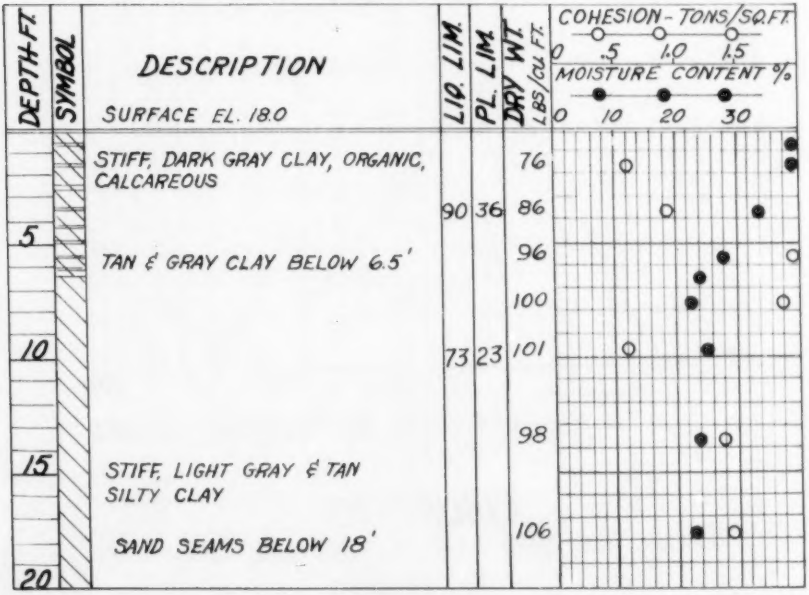

OIL STORAGE TANKS
EFFECT OF DIFFERENTIAL SETTLEMENT
UPON GAGING ACCURACY
1.0
.10
.01
.001
GAGING ERROR - BBLS. PER FT. OF GAGE
1
10
100
DIFFERENTIAL SETTLEMENT - INCHES

FIGURE 10

CORE BORING No. 1



CORE BORING No 2



CORE BORING No. 3

DEPTH - FT.	SYMBOL	DESCRIPTION	LIQ. LIM.	PL. LIM.	DRY WT. LBS/CU. FT.
		SURFACE EL. 27.6			
5		PLASTIC TO STIFF, GRAY & TAN CLAY, ORGANIC	68	30	84
					83
			76	31	85
10		PLASTIC TO STIFF, TAN & LIGHT GRAY CLAY, CALCAREOUS - VERY SANDY & SILTY BELOW 10 FT.			87
					106
					102
15		FIRM, TAN & LIGHT GRAY SILTY SAND WITH CLAYEY SILT & SILTY CLAY SEAMS & STREAKS.			103
20		WATER LEVEL AT 6.5 FT. DEPTH			100

COHESION - TONS/SQ. FT. 0 .5 1.0 1.5

MOISTURE CONTENT - % 20 30 40

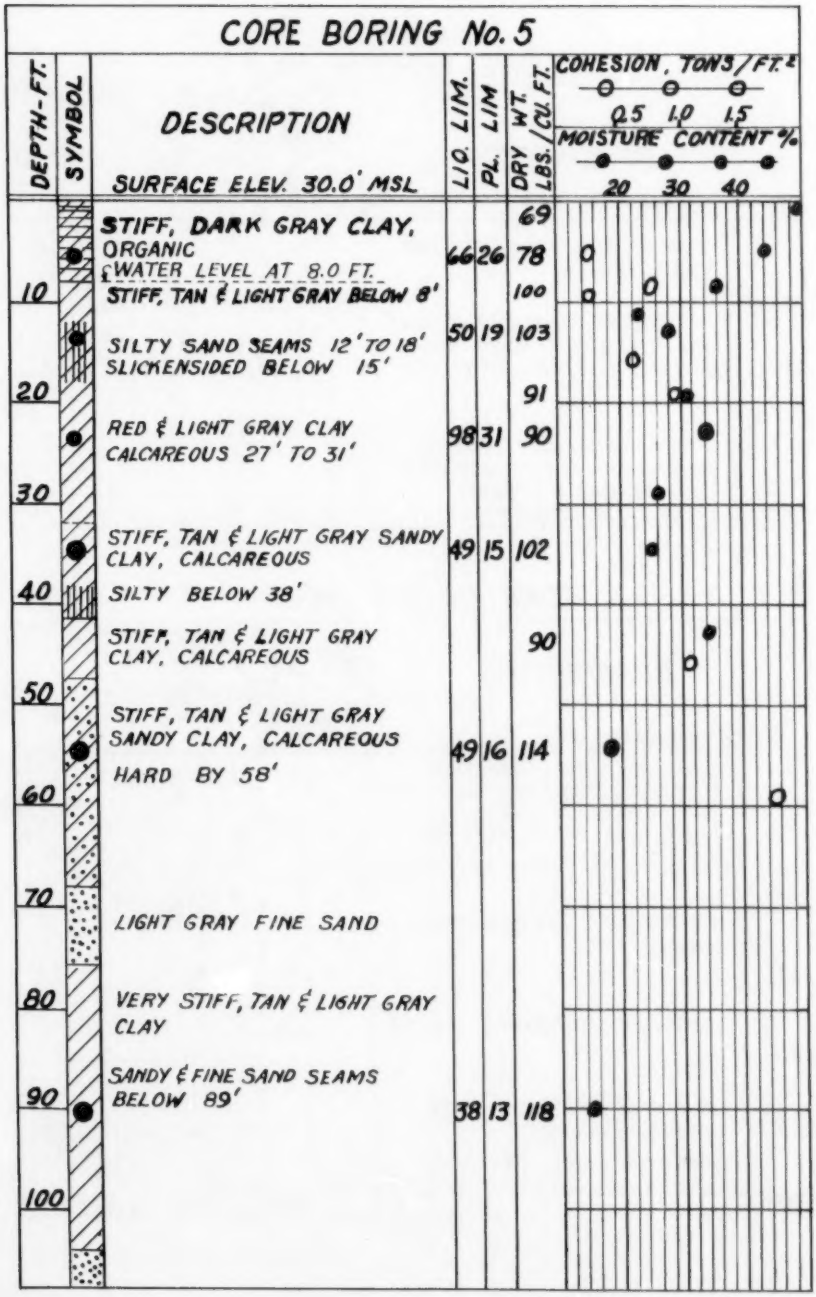
CORE BORING No. 4

DEPTH - FT.	SYMBOL	DESCRIPTION	LIQ. LIM.	PL. LIM.	DRY WT. LBS/CU. FT.
		SURFACE EL. 25.8			
		GRAY & TAN, CLAYEY, FINE SAND			
5		STIFF, GRAY & TAN SANDY CLAY WITH SAND SEAMS TO 4 FT.			120
		TAN & VERY SANDY AT 6 FT.			
10		TAN, SLIGHTLY CLAYEY, FINE SAND			
		LIGHT GRAY & TAN, SILTY, FINE SAND			
15					
20		VERY STIFF, LIGHT GRAY & TAN CLAY			106
25		LIGHT GRAY & TAN, FINE SAND WITH CLAY SEAMS			
		VERY STIFF, GRAY & TAN CLAY, FERROUS NODULES - SAND 26' TO 27'			
30					105

COHESION - TONS/SQ. FT. 0 .5 1.0 1.5

MOISTURE CONTENT - % 20 30 40

CORE BORING No. 5



Paper 1864 December, 1958 SM 5

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SOIL MECHANICS AND FOUNDATIONS DIVISION
Proceedings of the American Society of Civil Engineers

DESIGN AND CONSTRUCTION OF THE AMBUKLAO ROCK FILL DAM[a]

E. Montford Fucik,[1] M. ASCE and
Robert F. Edbrooke,[2] A.M. ASCE
(Proc. Paper 1864)

SYNOPSIS

The problems of designing and constructing a 430 foot high, vertical core rock fill dam are described in this paper. As nearly as can be determined, this is the highest dam of this type in the world now operating at maximum reservoir level. The Ambuklao project consists of, in addition to the dam, a chute spillway, underground powerhouse, and low level outlet works. The river was diverted through three tunnels in the right abutment of the canyon, two of the tunnels being converted for use as a power tunnel and low level outlet tunnel after diversion.

Field exploration and sampling, coordinated with laboratory and field testing provided a sound basis for the selection and design of the dam.

Records of settlement and seepage after operation at maximum reservoir bear out the adequacy of design and construction procedures.

General Description

The Ambuklao Rock Fill Dam is part of the Ambuklao Hydroelectric Project, located on the Agno River, Island of Luzon, Republic of the Philippines. The Project is owned and operated by the National Power Corporation, a Philippine government owned corporation whose function it is to develop hydroelectric power projects throughout the Republic. The total cost of the Project was about 132,000.000 pesos ($66,000,000), of which about $20,000,000 was loaned by the Export-Import Bank of the U. S. The remain-funds, 92,000.000 pesus ($46,000,000) were provided by the Philippine Government.

Note: Discussion open until May 1, 1959. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1864 is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. SM 5, December, 1958.

a. Presented at the June, 1957 ASCE Convention in Buffalo, N. Y.

1. Executive Vice Pres., Harza Engineering Co., Chicago, Ill.
2. Asst. Project Mgr., Harza Engineering Co., Chicago, Ill.

The construction of the Project was started in 1952, and the first generating unit was put in operation in December, 1956.

Figure 1 shows the location of the Project in relation to Manila, where the bulk of the power generated at Ambuklao is used. It can be seen that the Project is about 135 miles from Manila.

The Agno River has a drainage area of about 265 square miles above the damsite. The flow regimen of the river has a distinct dry season and a wet season. As can be seen in the river hydrographs in Figure 2, the dry season extends from about December to April, and the wet season from May to November. This well defined runoff pattern, typical of most areas in the Tropics, was very influential in determining the pattern of the construction procedures on the dam.

The maximum flood of record at the damsite was estimated to have had a peak daily flow of about 210,000 cfs, which is equivalent to a runoff of about 800 cfs per square mile of drainage area. This high runoff is not surprising when it is realized that a rainfall of 48 inches in 24 hours has been actually recorded at Baguio City about 12 miles from the site! The spillway design flood for the Project is 390,000 cfs.

Figure 3 is a plan of the major structure of the Project. The rock fill dam is placed at a narrow point in the river valley, and the spillway is located at the left end of the dam. This spillway location takes advantage of the return loop made by the river downstream of the dam, and thus the discharge from the spillway enters the river about a mile downstream from the toe of the dam, riverwise, but only 500 feet away measured in a straight line.

The spillway, as shown in Figure 3 and Figure 4 (cross-section) is a high chute section, controlled by seven tainter gates, each 12.5 x 12.0 meters. The gates are operated by individual motor operated gate hoists, controlled from the power station. The shape and elevation of the spillway bucket was established as a result of model tests carried out at the hydraulic laboratory of the University of the Philippines. The bucket is proportioned to throw the water out from the slope below the bucket into the river. Observations of the flows passed over the spillway so far indicate good correlation between the model results and the prototype.

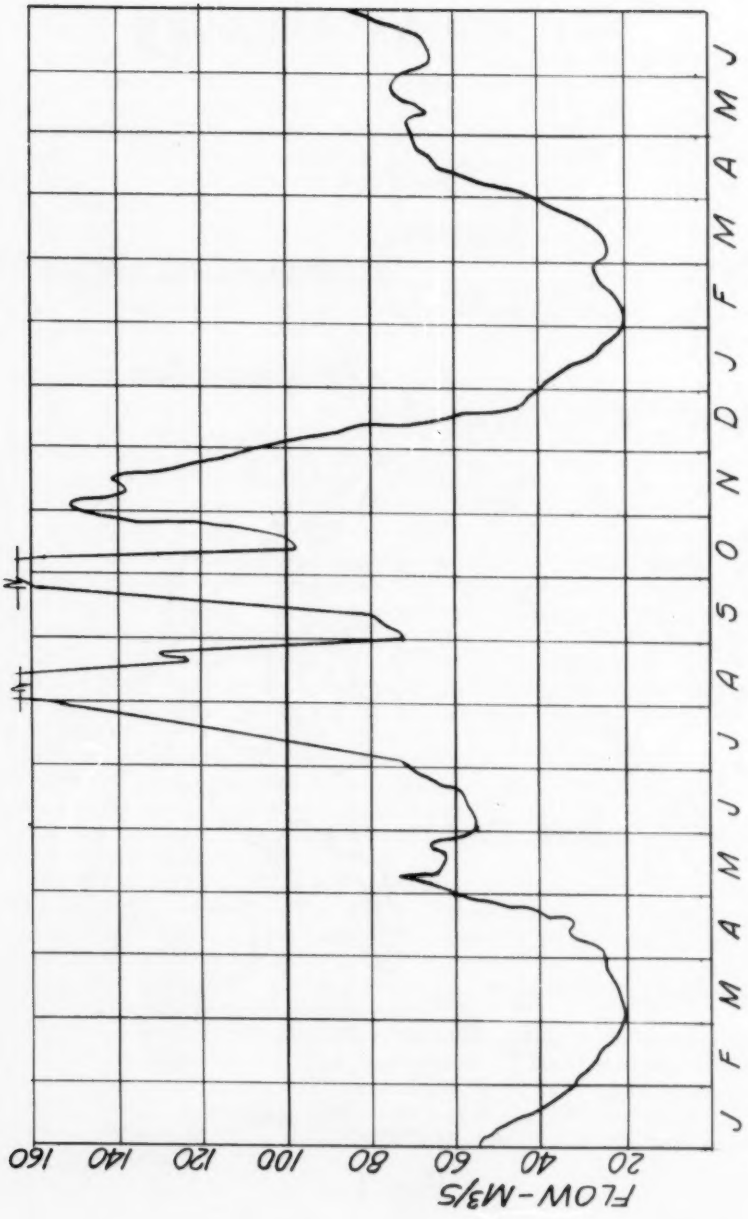
Diversion through power tunnel "C" also takes advantage of the large loop in the river. By running this tunnel underneath the river at the dam axis, and into the river at the point where the loop returns, an additional head of about 60 feet was possible, resulting in a saving in tunnel size for passing the flows during the diversion period.

The power station is located in an underground cavern in the right abutment, and here also advantage is taken of the sinuous course of the river. As is shown on Figure 3, the tailrace tunnels from the power station runs under two bends in the river and exits to the river about 7200 feet from the power station. The 180 feet of additional gross head obtained by this length of tail tunnel proved well justified economically.

The water for the turbines enters through an intake tower in the reservoir, located at the upstream end of the power tunnel "C." This tunnel is utilized to carry the flow to a point over the power station, where it is divided into three penstocks. Each penstock has two valves ahead of the turbines. There are three generating units, operating at a normal head of 505 feet, and each generating 25,000 kw of power.

PHILIPPINE ISLANDS
N
LUZON
BAGUIO
AMBUKLAO PROJECT
Philippine Sea
MANILA
China Sea
Sulu Sea
0 100 200 Miles

Fig. 1



TYPICAL HYDROGRAPH
AGNO RIVER - SAN MANUEL Sta.

Figure 2

AGNO RIVER
Quarry #1A
Spillway
Tail Tunnel
AMBUKLAO
POWER PROJECT
0 500 Feet
Figure 3
Low Level Outlet Tunnel #2
Diversion Tunnel #1
Power Tunnel #1
Quarry #2
N
650
700
750
800
0 1000 feet

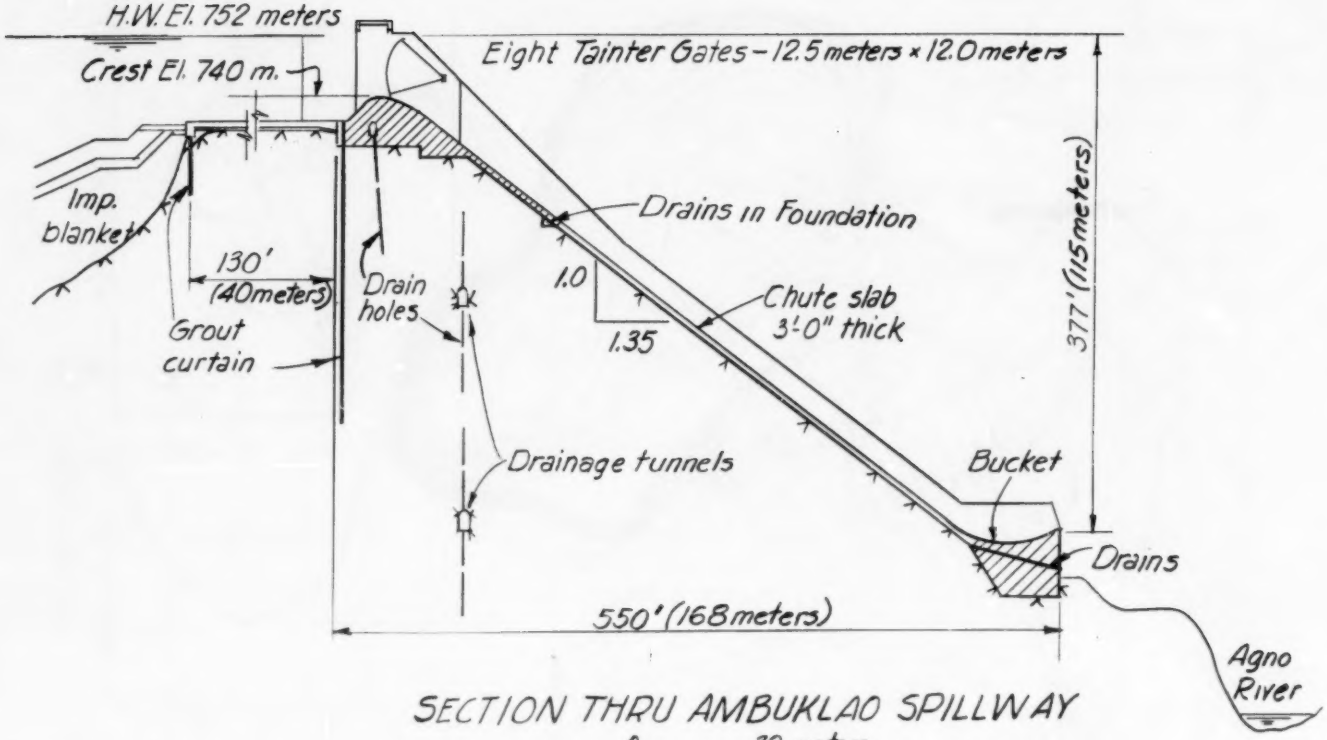

H.W. El. 752 meters
Crest El. 740 m.
Eight Tainter Gates – 12.5 meters × 12.0 meters
Imp. blanket
130' (40 meters)
Grout curtain
Drain holes
Drains in Foundation
1.0
1.35
Chute slab 3'-0" thick
377' (115 meters)
Drainage tunnels
Bucket
Drains
550' (168 meters)
Agno River
SECTION THRU AMBUKLAO SPILLWAY
0 30 meters

Figure 4

Geology of the Site

At the Ambuklao damsite the river flows from north to south in a narrow gorge between two ridges, the western ridge being composed of metamorphic rocks and the eastern ridge of both metamorphic rocks and diorite. Above the gorge the river is in a relatively wide valley which will form the reservoir basin. Below the gorge the river flows in a narrow meandering valley.

The principal rocks of the Agno Valley in the vicinity of the Ambuklao damsite are diorite and a series of metamorphosed andesite, tuffs, lavas, and sedimentary rocks. The members of this series were laid down as lava flows and water deposited sediments in Tertiary times. After deposition when those rocks were buried deep beneath the surface they were subjected to mountain building forces which produced numerous fractures and incipient fractures. The lavas and sediments were metamorphosed and their original characteristics altered. While still buried far beneath the surface these metamorphosed lavas and sediments were intruded by molten diorite. As a result the contact of the diorite and the metamorphosed rock is extremely irregular but it is practically everywhere a tight, fused contact. Both the metamorphics and the diorite have been intersected here and there by fault or shear zones. The entire series of metamorphosed lavas and sediments is referred to as the metamorphics. There is also in the right abutment ridge far back from the end of the dam an elongated intrusive body of granodiorite.

The rocks of this region are deeply weathered as is true practically everywhere in the humid tropics, therefore, hard unweathered rock is seldom exposed except where there has been geological recent erosion. Exposures of weathered rock give little indication as to the true characteristics of the unweathered rocks. Below the zone of weathering, however, the rocks are hard, though brittle, and the fractures are generally tight. This is illustrated at the Ambuklao damsite where the metamorphic rocks of the left abutment ridge are deeply weathered and appear to be in very bad condition but where seen in tunnels, these metamorphic rocks are hard and generally tight.

The geologic conditions at damsites in the tropics are in many ways opposite to those found at damsites in cold countries. In the tropics the rocks in the abutments are practically always deeply weathered and in poor condition, but the rock under the river bed is usually in good condition and the depth to bedrock is generally not excessive. In the glaciated regions of the northern countries, however, the abutments are generally in good condition with sound rock at or near the surface, but under the river the depth is often excessive. This contrast of conditions at damsites in the tropics and in northern regions must be taken into consideration in comparing damsites in the tropics with those with which American engineers are more familiar.

Figure 5 is an areal geologic map of the damsite. About 150 diamond core drill holes were drilled in the damsite area (not all indicated on Figure 5), to determine the geologic and engineering characteristics of the underlying rock. Based on the evidence from these borings and surface observations, it was concluded that the rock in the abutments and in the river bed was not sufficiently strong to withstand the pressure which would be developed by any economically feasible type of concrete dam. The presence of satisfactory quarry rock in the valley sides near the damsite led to the selection of a rockfill dam. Abundant clayey material for use in the core was also available near the tops of the adjacent hills. A geologic section along the axis of the dam is shown in Figure 6.

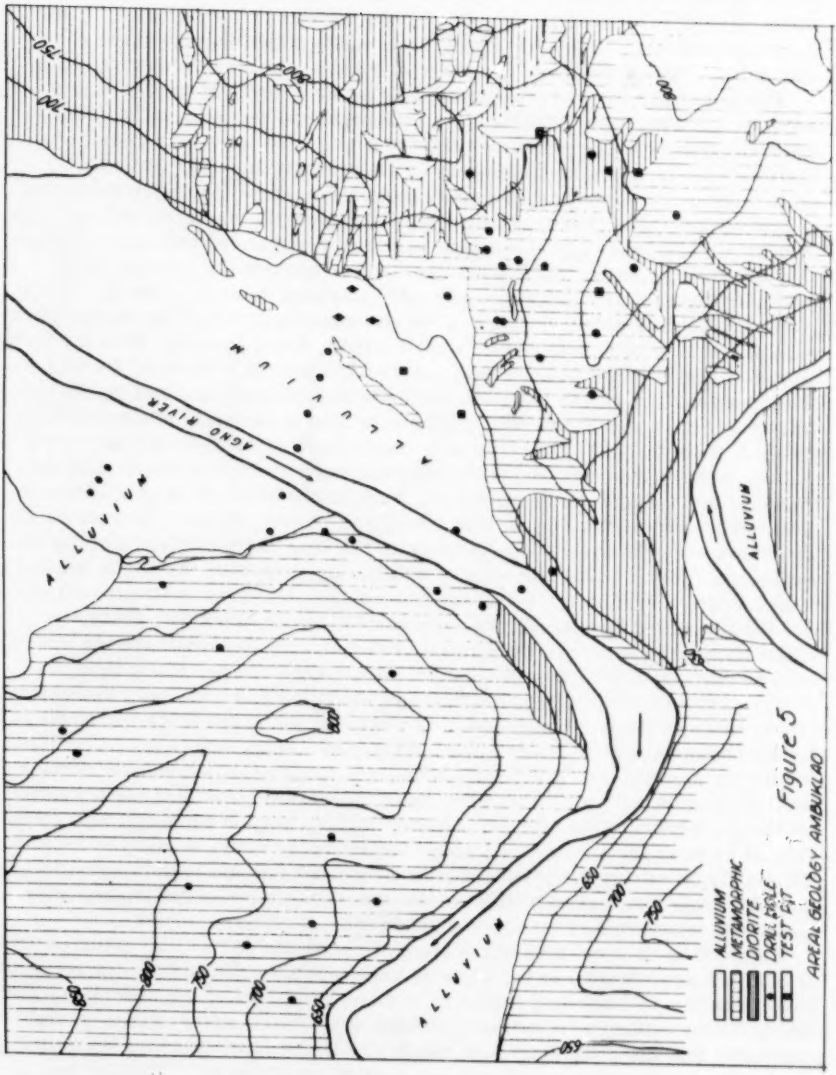
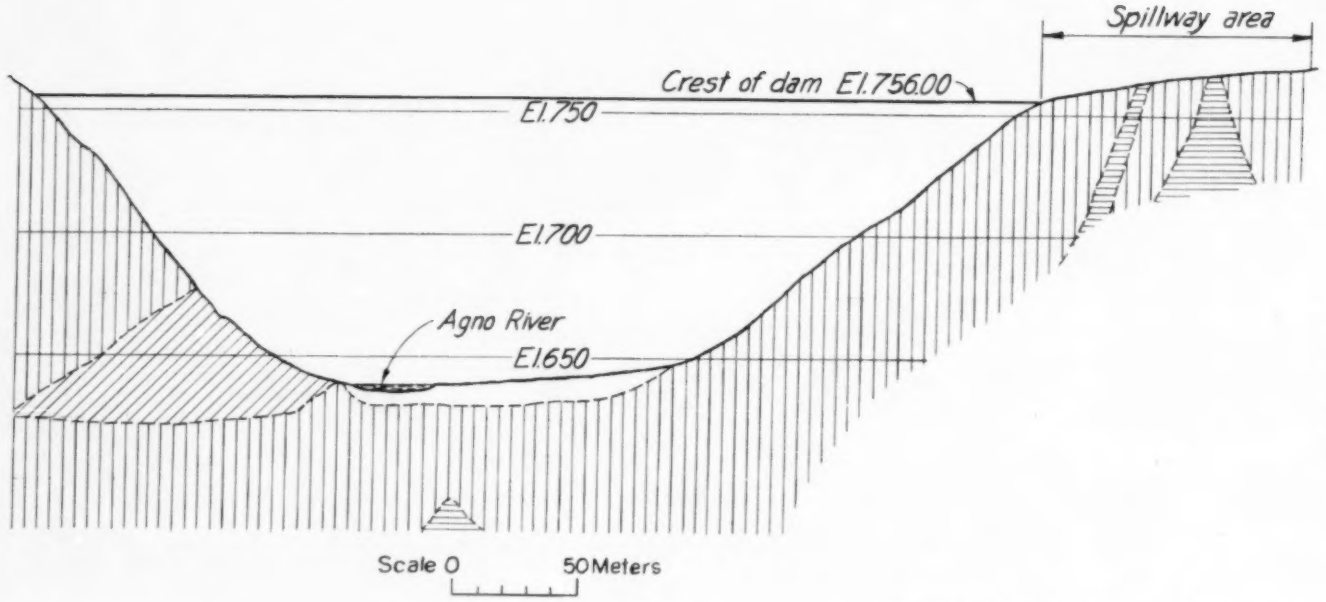

ALLUVIUM
AGNO RIVER
ALLUVIUM
ALLUVIUM
ALLUVIUM
ALLUVIUM
METAMORPHIC
DIORITE
DRILL HOLE
TEST PIT
AREAL GEOLOGY AMBUKLAO

Figure 5



GEOLOGICAL SECTION
AMBUKLAO DAM

Figure 6

Testing Program

To determine the suitability of the rock and clay material for use in the shells and core of the dam a rather extensive series of exploratory drifts and test pits were carried out.

In an effort to check the adequacy of the rock in the proposed quarry pits for use in the rockfill, a series of crushing tests were made in the laboratories of the National Power Corporation. These tests were concentrated on materials from quarries 1A and 2 indicated on Figure 3. These quarries were located in areas most convenient for construction.

Quarry 1A was primarily diorite and initial exploration indicated that a small amount of overburden should be expected. Below the overburden the large mass of diorite was present and samples recovered from the drill holes and exploratory drifts indicated that sharp angular rock could be expected from the quarry blasts. Quarry No. 2 was composed of a large mass of metamorphic rocks. This quarry was ideally located for the downstream rockfill and indicated a material suitable for the shell.

The crushing tests were carried out by placing samples of the broken rock from the quarry areas in a container and loading the surface of the sample with a load equal to the pressure which would be exerted on the bottom layer of the rockfill by the overlying fill material. The container was a 30" diameter cylinder about 20" deep and using a hydraulic jack and loading plate 10 inches square a pressure of about 25 tons per square foot was developed on the material in the box. The settlement under the load as shown in Figure 7 and the gradation curves of the rock before and after the test indicated how much the rock might break down under the pressures to be expected in the dam. Prior to this test the general expectations were that the diorite would prove to be much stronger than the metamorphic rock. The results were so contrary to what had been expected by visual inspection of the

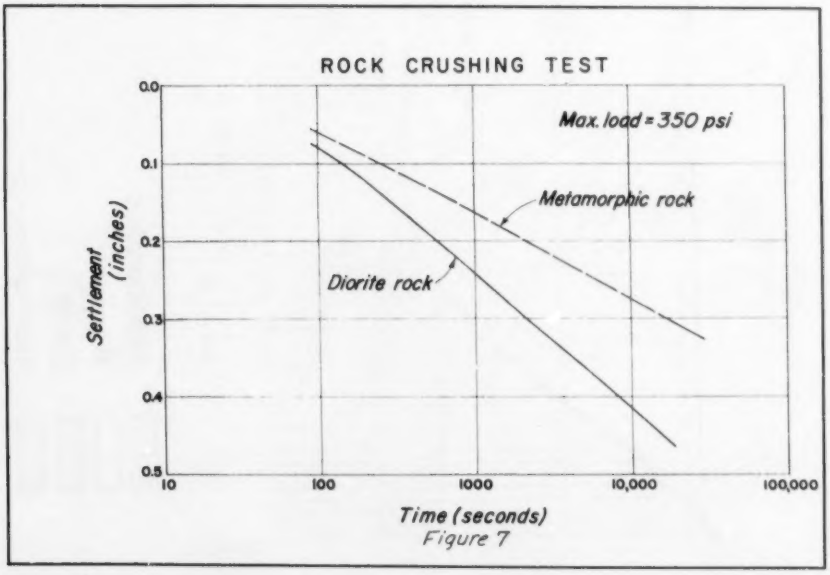


Figure 7

two rock types that at first it was suspected that the results had been reversed. However, the breakdown of the diorite during blasting, handling, and placing in the dam was much greater than the metamorphics, indicating the value of such tests as an indicator of the relative values of different rocks for use in rockfill.

In addition to the rock crushing tests, the more usual tests of compaction, grading, triaxial and consolidation tests were made on the core material from the borrow pit areas. Examination of the borrow pits disclosed that high moisture contents could be expected throughout the year. The natural moisture contents ranged from 5 to 10% above the optimum moisture content. The long rainy season and high humidity (even in the dry parts of the year) were the prime factors causing this condition.

Three field test fills using the impervious borrow materials were made under conditions simulating those expected during construction of the dam. Laboratories of the National Power Corporation tested samples of the test fill to determine compaction, shear strength and permeability. Additional samples we sent to the USA for confirmation tests and triaxial shear tests since the National Power Corporation did not have access to a triaxial machine.

Approximately ten quick triaxial series were made in the U.S. at varying moisture contents. The triaxial tests were run at 1, 3 and 5 kg/cm^2, which was considered sufficient to establish the trend of the materials' characteristics when coordinated with consolidation and permeability tests. The tests indicated that high pore pressures could be expected at moisture contents above the optimum. Armed with this information the field testing program was extended to determine if there was a possibility of reducing the moisture content by additional scarifying of the borrow material. The scarification proved to be a long and expensive procedure of dubious effect, in fact, a reduction of the moisture content 5% required two or three days of scarifying. Therefore, the design was predicated upon having high pore pressures in the core.

Design

Preliminary layouts, stability studies, and quantity take-offs were made of three types of dams:

1. Large earth fill core with rock shells
2. Sloping core rock fill (Fig. 8)
3. Narrow center core rock fill (Fig. 9)

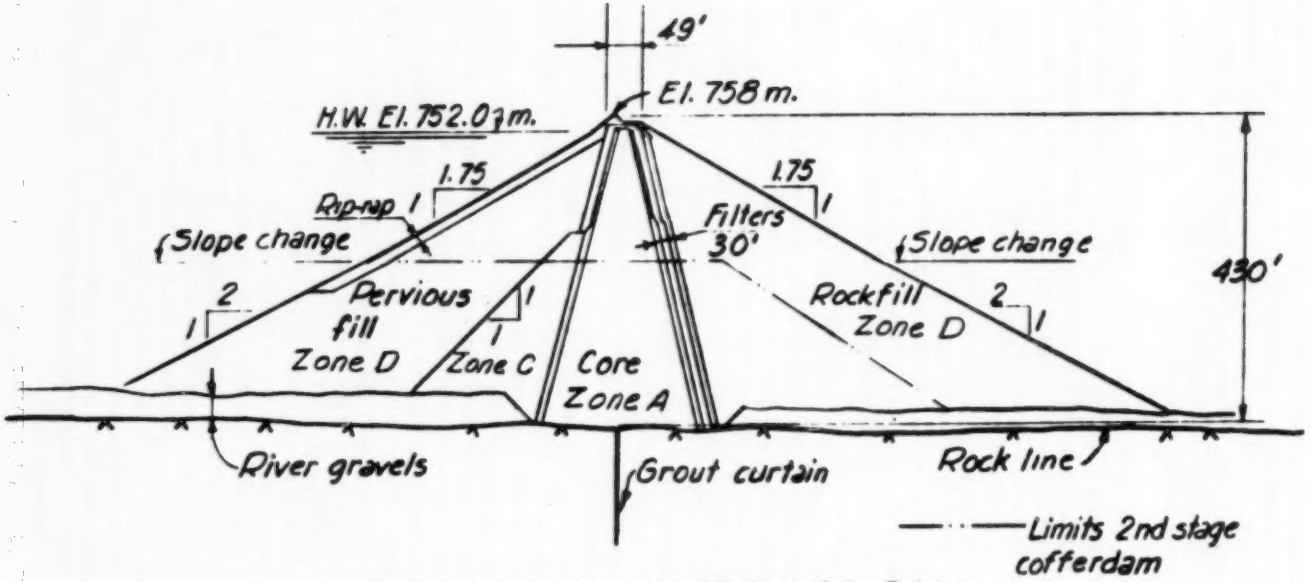
These design studies indicated that a dam with a large center core of earth would require flatter slopes for stability than either of the rockfill types. Also the earthen fill presented a difficult construction problem because of the high humidity and rainfall at the site. The choice then narrowed down to a sloping core or vertical core rock fill.

The design studies indicated that a rockfill dam with a vertical core could be built with less quantities of material than a sloping core rockfill. Referring to Figure 3 it can be seen that the downstream shell of the dam is at a much narrower section of the canyon than the upstream section. A sloping core dam permits a steeper downstream slope than for a vertical core, but requires some flattening of the upstream slope because the core is located in an area of high shear stress, and thus requires more rock shell for stability.

SECTION THRU SLOPING CORE DAM (alternate study)

0 50 meters

Figure 8



SECTION THRU AMBUKLAO DAM

0 50 meters

Figure 9

Flattening the upstream slope increased the quantities more than the saving made by steepening the downstream slopes. This was a major factor in the deciding to use a central core type dam. In addition, the increased contact area between the vertical core and the rock foundation gives a longer seepage path under the core than could be obtained with a sloping core. This is greatly to be desired under the some 400 feet of head to which Ambuklao is subjected.

Another feature of the vertical core design which led to its selection was the fact that the full weight of the dam is applied to the contact area between the core and the foundation, thus giving the maximum protection against possibility of leakage along the contact. The pressure along the contact surface of the sloping core, at the foundation level, is considerably less.

The stability of the dam was analyzed by two basic theories of design, the "sliding-wedge" and "slip-circle." The "sliding-wedge" theory we consider the most applicable to the analysis of a structure where in the most critical plane of stability is located predominantly in a frictional, non-cohesive material. Conversely, when the most critical plane of stability passes through a cohesive material of little or no frictional strength, the "slip-circle" is the most applicable.

The vertical core type of rockfill dam is best analyzed by the "sliding-wedge" method. Figure 10 shows an example of this analysis for the construction condition with maximum pore pressure development.

The "slip-circle" analysis was applied to the left abutment where the impervious blanket (discussed below) formed a major part of the dam section. Also, as a matter of record, the "slip-circle" was used as an independent check of the sections analyzed by the "sliding-wedge" theory. As expected, the slip-circle gave greater safety factors because only a small part of the circle passes through the weak core material, and thus does not reflect the effect of high pore pressures.

The most critical zones of stability for this dam were, (a) the maximum section in the canyon, (b) the left abutment and (c) in the area of the fault zone which existed below the upstream shell.

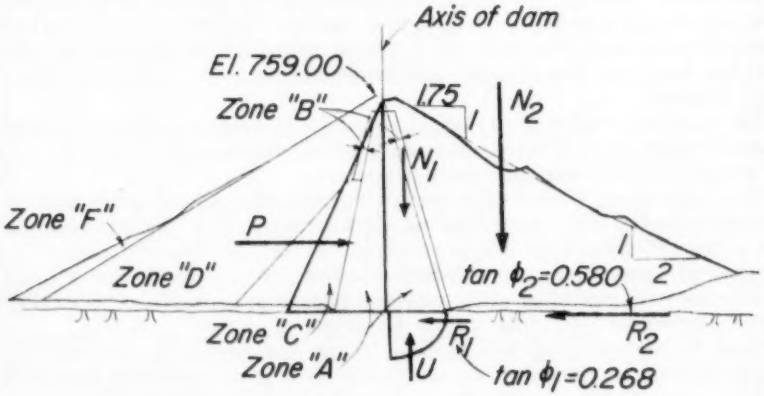
The narrow ridge that formed the left abutment was analyzed from a standpoint of structural stability and seepage. The weathered material of the narrow ridge was assumed to be of low shear strength, (a value of 17 degrees was considered quite conservative) and the stability was checked by use of the sliding wedge analyses. In order to reduce seepage and provide a better cut-off in the left abutment, an impervious blanket was placed on the upstream face of this narrow ridge and was continuous with the main core of the rock-fill dam as partially shown on Figure 4.

A fault zone extended diagonally across the canyon and on the left bank formed the valley immediately upstream of the spillway. A portion of the upstream shell was founded on this fault. Examinations revealed that the fault zone was a fairly impervious mass throughout. Potential planes of leakage were assumed to exist at the edges of the fault zone where the parent rock contacted the brecciated material of the fault and necessitated additional treatment. The left abutment blanket, mentioned above, was laid out to provide additional cutoff and reduce the hazard of excessive seepage through this fault zone. Fortunately the fault zone was upstream of the highest foundation shear stress area so that a trench, backfilled with rock, provided the necessary stability to the upstream shell.

Condition I – Construction

Pool empty (max. pore pressures)

SECTION I-I (Sta. 4-933.669)



Driving forces:

$P = 1440^{K}$

$U = 421^{K}$

Resisting forces:

$R_1 = (N_1 - U)\tan\phi_1$ $\quad R_1 = 189^{K}$

$R_2 = N_2 \times \tan\phi_2$ $\quad R_2 = 1832^{K}$

2021^{K}

Factor of safety:

$$F.S. = \frac{2021}{1440} = 1.40$$

Min F.S. = 1.21 (Earthquake)

Figure 10

Except for the foundation of the core, the existing river gravel was left in place. The gravels were clean and for conservative design assumptions, in keeping with a dam of this magnitude, a value of 30° was used as the angle of internal friction in all stability analyses.

To check the stability under the most severe conditions of pore pressure rise and the corresponding loss of strength of the core material the core is assumed to act as a fluid mass without shear strength. The minimum safety factor of the maximum section, under earthquake, was calculated by the sliding wedge analysis to be 1.21. Even if this severe condition did exist the safety factor would greatly increase as consolidation occurred and pore pressures were relieved.

The crest of the dam was constructed about 3.25 feet above the design elevation to provide for future settlement. With close control during placement of the central core and the rolling and spreading and sluicing of the upstream shell, it was felt that settlements in this structure would be small. Measurements of settlement after bear out this assumption in that the vertical settlement has been less than 6 inches and the downstream movement has been about 3 inches.

For a structure of this magnitude and range in grain size from the fine-grained central core to the coarse and massive structure of the rockfill shell, the design of filters was extremely important.

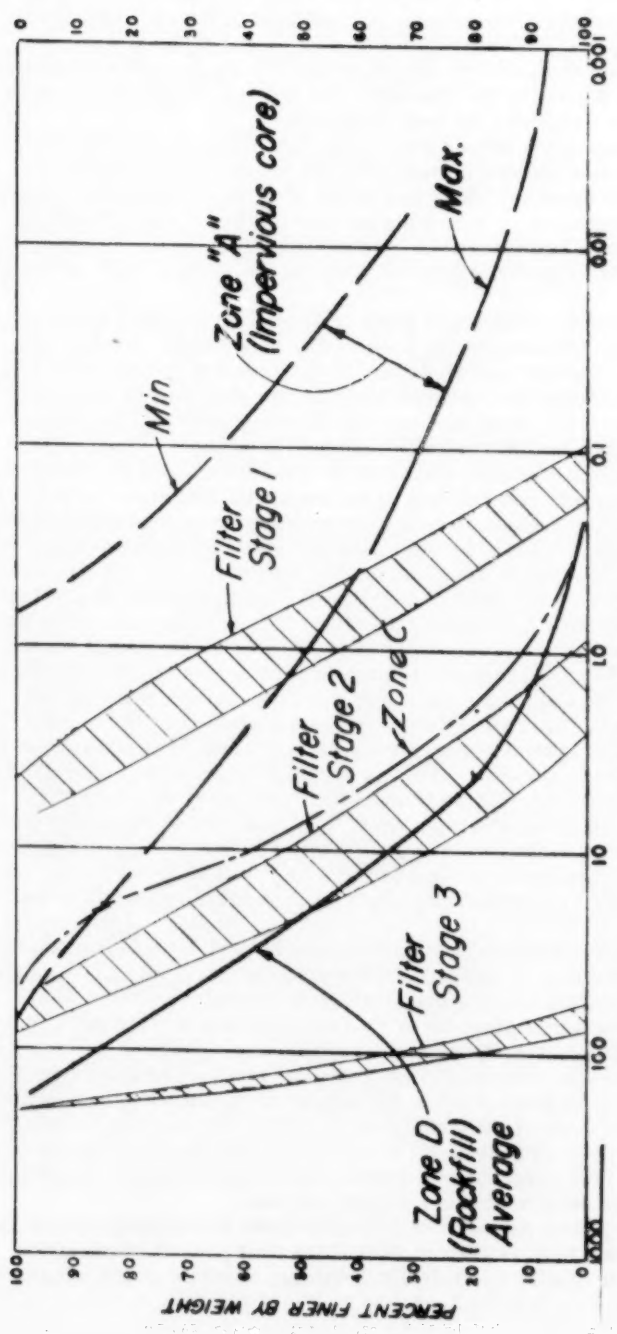
The basis of our filter design was predicated on past experience and laboratory test of filter materials on other projects. The grain size relationship of the 15% size of the coarser material to the 85% size of the finer material was specified to be less than a factor of five (5). Also the gradation of each material should be sufficiently uniform to prevent piping or migration of particles within the limits of each material.

There were three stages or types of filters provided on the downstream face of the vertical core and two stages provided on the upstream face of the vertical core. The transition on the upstream side of the core could be accomplished in two stages of filter. The finer grained upstream shell made this possible, however, on the downstream side of the core the range in gradation required the addition of a third stage of filter to prevent possibility of migration of either the core materials or the filter materials into the downstream rockfill shell. The gradation of the filters is as shown on Figure 11. The gradation of these filters could be made comparable to that required for the concrete aggregates. This permitted some economy of production and reduced the number of operations required to produce filter materials and concrete aggregates.

The freeboard provided to protect against overtopping by wave action under typhoon winds was about 13 feet. Although the freeboard is about 19.5 feet at normal operating pool elevations the design freeboard is measured from the surcharged pool elevation that would occur during the event of a design flood. To minimize the cost of the allowance for freeboard, a hand-placed rock parapet was provided above the roadway to compensate for the surcharged pool condition. The calculation of the anticipated wave height equated the fetch, average depth of pool, and wind velocities with adjustments for the effect of wave impact on a sloping surface.

Construction

The contract for the construction of the dam, spillway, and diversion tunnels was signed in January 1953 with Guy F. Atkinson Company of



GRAIN SIZE CURVES

Figure 11

San Francisco. The contract was on reimbursable cost basis with the fee related to the actual costs of the project as compared to the estimated costs, adjustments being made for variation of quantities and scope of work.

The 25 mile access road from Baguio to the site was partially completed at the time of the signing of the contract. The National Power Corporation and the contractor completed the road to the site.

After completion of the access road to the site the contractor initiated work on the diversion tunnels, cofferdams, and preparation of the quarry sites. To pass the expected floods that might occur during the rainy season, three diversion tunnels were scheduled for construction. The cofferdams, both upstream and downstream were located beyond the limits of the impervious core, and where possible within the limits of the rockfill shell as shown on Figure 9.

The first large quarry blast was made at Quarry 1A and gave results that were not expected. The weathering process has progressed deep into the hillside, in addition to the fractures that had developed in geologic history. Therefore, upon blasting the particles ranged from sand size to large boulders, approximately 60% were smaller than coarse gravel and, therefore, not suitable for the usual dumped rockfill construction methods. Grain-size curves of this rockfill material are shown as Zone D material in Figure 11. Since this rockfill shell material was at variance with that assumed in the initial design stages, large scale triaxial tests were immediately scheduled through the cooperation of the United States Bureau of Reclamation Laboratory in Denver, Colorado. Due to the urgency of these tests, about 500 pounds of material were flown by air express from the Philippines to Chicago and then transported to Denver for the testing program. The triaxial tests were run on representative samples of the material from Quarry 1A. Lateral pressures as high as 75 psi were used to simulate loading conditions anticipated in the prototype. Basically, the results of this program indicated that the outside dimensions of the dam would not require a revision. The frictional strength of this finer-grained quarry material was equal to or better than that assumed in the design stages. Frictional shear strength in the order of 40 to 45 degrees were obtained in these triaxial tests. In order to reduce the amount of waste within the upstream shell, Zone C shown in Figure 9, was the finest grained material and had the least structural strength, friction angles were 35^{o} for design purposes. Placed in this area just upstream of the core and below Elevation 710, it has little effect on the ultimate strength of the rockfill dam.

On the upstream side of the dam, cofferdamming was done in two stages. The first stage was a low cofferdam, built during the dry season, to turn the river into the 13 foot diameter circular diversion tunnel "A." This was necessary to permit a full clean-up of the fault zone that was located under the upstream shell. By the advent of the second rainy season of the construction period a diversion capacity of 74,000 cfs was required to assure reasonable safety from overtopping. The higher second stage cofferdam was incorporated within the limits of the rockfill shell. The core was also raised during this period and the downstream shell was built to lines that would afford temporary stability to the core. In this way the maximum amount of equipment could be utilized in construction of the upstream shell.

A late season typhoon named "Cora" caused flows that overtopped the first stage cofferdam late in November of 1953 of the first year of construction. Repairs were made quickly and very little damage was done to the existing

construction and little time was lost. For the remainder of the construction program there was little difficulty in diverting the river.

Tunnels "B" and "C" were designed for passing flood flows. Tunnel "B" was a 25-foot diameter horseshoe tunnel and tunnel "C" was a 23-foot diameter horseshoe tunnel, both were concrete lined throughout their length. After diversion tunnel "C" was converted into the power tunnel and the previously mentioned 13-foot tunnel "A" became the low level outlet tunnel. Tunnel "B" was plugged after the diversion period, having served its function for the construction of the project.

The preparation of the foundation for the core and the shell was started at the abutments prior to the closure of the river, then after closure the preparation of the central portion was completed. The methods of excavation were both hydraulic and by mechanical means as shown in Figures 12 and 13. Portions of the area under the core where deep depression existed were backfilled with concrete. In general the use of hand tamped impervious fill was all that was necessary to bring the foundation to a level surface where the mechanical equipment could function.

The grout curtain for the dam is shown in Figure 14. The deep cutoff grout curtain was designed to reduce uplift and increase the seepage path so that

FIGURE 12
SLUICING LEFT ABUTMENT

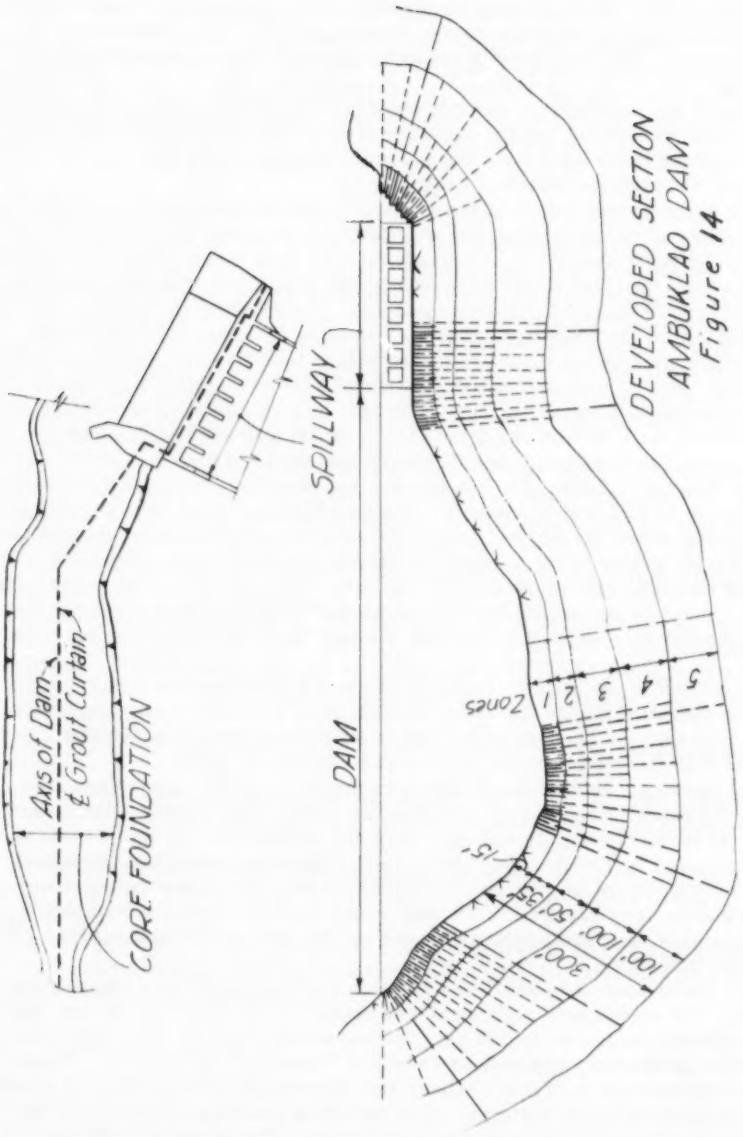
FIGURE 13
CORE FOUNDATION EXCAVATION

gradient would be low and thereby reduce the danger of blow outs and/or excessive seepage volume.

The full depth of the grout curtain was developed by the stage grouting method, with a thorough washing and pressure testing prior to grouting. As each zone was grouted the permissible pressure was increased for next lower zone. The following table shows the pressure used:

Zone	Depth To	Depth From	Maximum Pressure	Design Spacing
1	0	15'	15 psi	5 ft. o.c.
2	15'	50'	50 psi	20 ft. o.c.
3	50'	100'	100 psi	20 ft. o.c.
4	100'	200'	200 psi	20 ft. o.c.
5	200'	300'	300 psi	200 ft. o.c.

The split spacing method was specified for all areas exhibiting large grout take and was to be a field adjustment. Packers were used to extend the grout


DEVELOPED SECTION
AMBUKLAO DAM
Figure 14
SPILLWAY
DAM
Zones
1
2
3
4
5
100'
100'
300'
50'
35'
15'
Axis of Dam
& Grout Curtain
CORE FOUNDATION

curtain for each successive zone. Split spacing is the procedure whereby a primary set of holes is drilled and grouted throughout a section for a depth zone at a prescribed maximum spacing and succeeding sets of holes are drilled and grouted located so as to split the interval between previously completed sets. This process is continued until the spacing is such that no appreciable amount of grout can be injected into the hole. The maximum spacing prescribed is shown in the table above in the column titled Design Spacing.

Zone 5 holes served as check holes to determine if any deep seated seepage channels existed and the pressure tests at 200 feet and 100 feet to the bottom of the holes confirmed the general tightness of the area and the effectiveness of the grout curtain.

The grouting operations in the lower portions of the canyon were done by drilling and grouting through the core material because of the pressure of time. The portion of the hole through the core was cased and the casing set in the rock. The abutments were drilled and grouted from the excavation line, prior to placing of the core material. The grouting in both cases was carried out by the stage grouting method, using packers. The holes were thoroughly washed and pressure tested before injecting the grout, and after grouting, deep check holes were drilled and pressure tested to establish the effectiveness of the grouting accomplished. A total of 72,500 lineal feet of grout holes were drilled and grouted with an average take of 0.3 bags of cement per lineal foot of hole. The split spacing for zones 2, 3 and 4 started at a 20 feet on centers. The closest spacing was 5 feet for zone 2, 10 feet for zone 3, and 20 feet for zone 4. The zone 5 check holes were pressure tested by packers set at each zone and finally grouted indicating the suitability of the completed grout curtain. In general the holes were very tight. The highest take was encountered in the spillway area and the lowest in the canyon floor. This was in keeping with the general geologic analogy put forth in the early part of the report, i.e. the best rock could be anticipated below river level.

Placement of the rockfill shell, filters and impervious core was a continuous operation except for short periods when the rains made core placement impossible. The maximum rate of placement was about 500,000 cubic meters (650,000 cubic yards) per month including all zones.

The upstream shell zones C and D were placed in 24" layers and compacted by rubber tired rolers and sluicing. The materials from the upstream quarries broke down quite fine and were not suitable for the more usual rockfill placement by end dumping and sluicing. Sluicing operations, however, were very effective in creating a compact mass. The 50-ton rubber tired rollers were very valuable in locating areas of below optimum compaction. In two passes, soft spots would be quite evident and remedial measures could be taken immediately.

The impervious core and filter materials were placed and compacted together. The compaction equipment was rubber tired rollers of 50-ton capacity with tire pressure of 80 to 100 psi. In general four passes on 18 inch layers provided satisfactory compaction in both the core and filter zones. Near the 75% completion point, it was realized that there was a possibility of a shortage of the first stage filter material. The specified thickness had been 10 feet for ease in placement by end dumping from trucks. This was reduced to 3-1/2 feet and placement was done by dumping into a sliding form with some hand tamping of the faces prior to placing additional core or stage 2 filter material. Figure 15 indicates the operation near the crest of the dam.

The downstream shell Zone D material was composed of large boulders grading down to smaller sizes. This material was suitable for placing by end dumping and sluicing as shown in Figure 16. The hydraulic monitors were directed so that they forced the fines down into the fill reducing the voids and thus keeping future settlement to a minimum. The lifts for end dumping varied from 30 to 90 feet, generally in the lower ranges.

The rip rap on the upstream face extended from below minimum drawdown level to the crest. A parapet of hand placed rockfill was provided above the roadway for protection from wave action and also service as a guard rail.

SUMMARY

The filling of the Ambuklao reservoir was started in the wet season 1955, and the reservoir was filled to elevation 740 M, spillway crest elevation, by early 1956. Observations of the completed structure indicate that it is performing satisfactorily. Very little seepage is evidenced through the left abutment ridge, indicating that the blanket on the upstream face of this ridge is performing well. Some seepage is still in evidence into the access tunnel and power station cavern in the right abutment and grouting is being done to seal this off. It appears probable that the grout curtain in the right abutment will have to be extended to provide additional cutoff.

The satisfactory performance of this high rockfill dam provides additional evidence that dams of this magnitude can be constructed economically and safely where a concrete dam would not be suitable, thus opening up for development damsites that might otherwise not be developed. Figures 17 and 18 are photographs of the completed project and figure 19 is a summary of significant data.

Mr. Fileman Rodgriquez, M. ASCE, was General Manager of the National Power Corporation during the construction. His assistant was Mr. F. M. Zablan, M. ASCE, who is now General Manager. Resident Engineer was Mr. Crispin Ubaldo, ASCE, and the Chief Construction Engineer was Mr. Ramon Ravanzo.

Guy F. Atkinson Company of San Francisco constructed the dam and spillway.

The geologic studies were made by Mr. Irving B. Crosby, ASCE, Consulting Geologist for the NPC, who followed this project throughout its development.

The liaison between the field and the design office of the Harza Engineering Company was provided by Mr. E. Rettig who was stationed in the Philippines.

Mr. E. Montford Fucik, Executive Vice President of Harza Engineering Company served as the Project Manager and Mr. R. F. Edbrooke was the Project Engineer in charge of design.

FIGURE 15
PLACING FILL NEAR CREST

FIGURE 16
PLACING DOWNSTREAM SHELL

FIGURE 17
GENERAL VIEW COMPLETED PROJECT

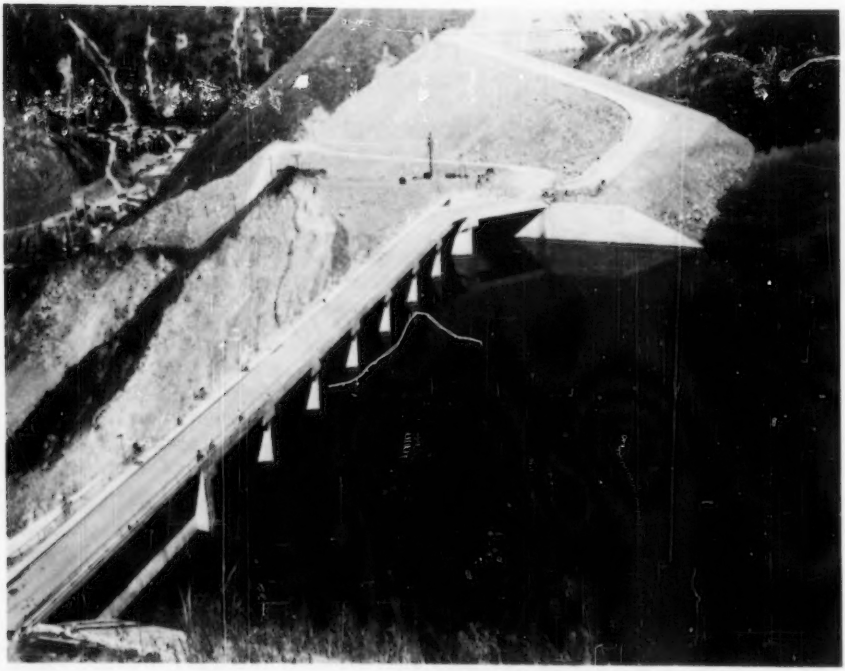


FIGURE 18
VIEW FROM LEFT ABUTMENT

SIGNIFICANT DATA

AMBUKLAO ROCKFILL DAM

Drainage Area	*265 sq. miles*
Length of Reservoir	*7.5 miles*
Max. Height of Dam	*430 feet*
Width at Base	*1455 feet*
Crest Length	*1480 feet*
Total Volume of Fill	*10,050,000 cu. yds.*

Material Characteristics

Zone A -	*ϕ 15°*	*Cohesion 500 psf*
Zone B -	*35°*	
Zone C -	*35°*	
Zone D -	*40°*	
River Gravels	*30°*	

Figure 19

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Journal of the

SOIL MECHANICS AND FOUNDATIONS DIVISION

Proceedings of the American Society of Civil Engineers

CONTENTS

DISCUSSION
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General Aspects of Cement Grouting of Rock, by V. L. Minear. (Proc. Paper 1145, January, 1957. Prior discussion: 1228, 1319. Discussion closed.). . . . . ---*

Increased Resistance to Deformation of Clay Caused by Repeated Loading, by H. B. Seed, R. L. McNeill and J. DeGuenin. (Proc. Paper 1645, May, 1958. Prior discussion: none. Discussion closed.)
by R. G. Ahlvin 1881-3
by D. Hugh Trollope 1881-6

Geological Factors in Tunnel Construction: Tools and Techniques, by Arthur B. Cleaves. (Proc. Paper 1648, May, 1958. Prior discussion: none. Discussion closed.)
by A. A. Eremin. 1881-9

Geological Factors in Tunnel Construction: Geology and Tunnel Design, by A. B. Reeves. (Proc. Paper 1649, May, 1958. Prior discussion: none. Discussion closed.)
by A. A. Eremin. 1881-11

The Structure of Compacted Clays, by T. William Lambe. (Proc. Paper 1654, May, 1958. Prior discussion: 1828. Discussion open until January 1, 1959.)
by D. P. Krynine 1881-13
by Dean R. Freitag 1881-16

The Engineering Behavior of Compacted Clay, by T. William Lambe. (Proc. Paper 1655, May, 1958. Prior discussion: 1828. Discussion open until January 1, 1959.)
by J. MacNeil Turnbull 1881-19

Note: Paper 1881 is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 84, SM 5, December, 1958.

*There will be no closure.

INCREASED RESISTANCE TO DEFORMATION OF CLAY CAUSED BY REPEATED LOADING[a]

Discussions by R. G. Ahlvin and D. Hugh Trollope

R. G. AHLVIN,[1] M. ASCE.—This is a very worthwhile research program and one likely to yield important new knowledge. In connection with the work reported, the writer wishes to call attention to several items from work of the U. S. Army Engineer Waterways Experiment Station which is, at least somewhat, related.

Fig. 1 is taken from a report of tests conducted on airfield pavement at Stockton, California, in 1942.(1) It shows a pattern (for the 40-kip load) similar to those in Figs. 6a, 7a, and 14 of the basic paper. Here, however, the measure of distress is in thickness of failing pavement rather than in axial strain. The break in this curve is caused by the distribution of test traffic over three adjacent wheel paths where prior traffic was in a single path. The pattern of the companion (25 kip) curve, which is for test traffic in three paths throughout, indicates that the break in the 40-kip curve would not have occurred if the early traffic had been distributed.

This leads to the question of the build up of residual stresses in a pavement construction or triaxial specimen. The residual stresses which can be built up by repeated application of a single stress pattern and which lend strength to the construction or specimen can be destroyed by a reversal of stresses. The authors should perhaps be wary of this phenomenon in their studies.

Fig. 2 shows field data comparable to the test data presented in the basic paper. These data are from a report of tests conducted at the Waterways Experiment Station in 1949 and 1950.(2) A test section consisting of three items of various strengths, all on medium-strength clay-silt subgrade, was subjected to loading by twin wheels in one lane and tandem wheels in another. In each case, 2000 coverages[2] of one loading were applied followed by 2000 coverages of a significantly heavier loading. Unfortunately, no comparable data are available for a test in which the larger loading might have been applied with no prior lighter load traffic. However, failures experienced in these test items under the heavier loadings rather than showing more strength than expected indicated that projections made from the pattern of prior proved design criteria were somewhat unconservative.

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a. Proc. Paper 1645, May, 1958, by H. B. Seed, R. L. McNeill and J. DeGuenin.

1. Engr. Chf., Special Projects Section, Flexible Pavement Branch, Soils Div., U. S. Army Engr. Waterways Experiment Station, Vicksburg, Miss.
2. A coverage consists of wheel passes in immediately adjacent paths, sufficient to cover the lane being trafficked.

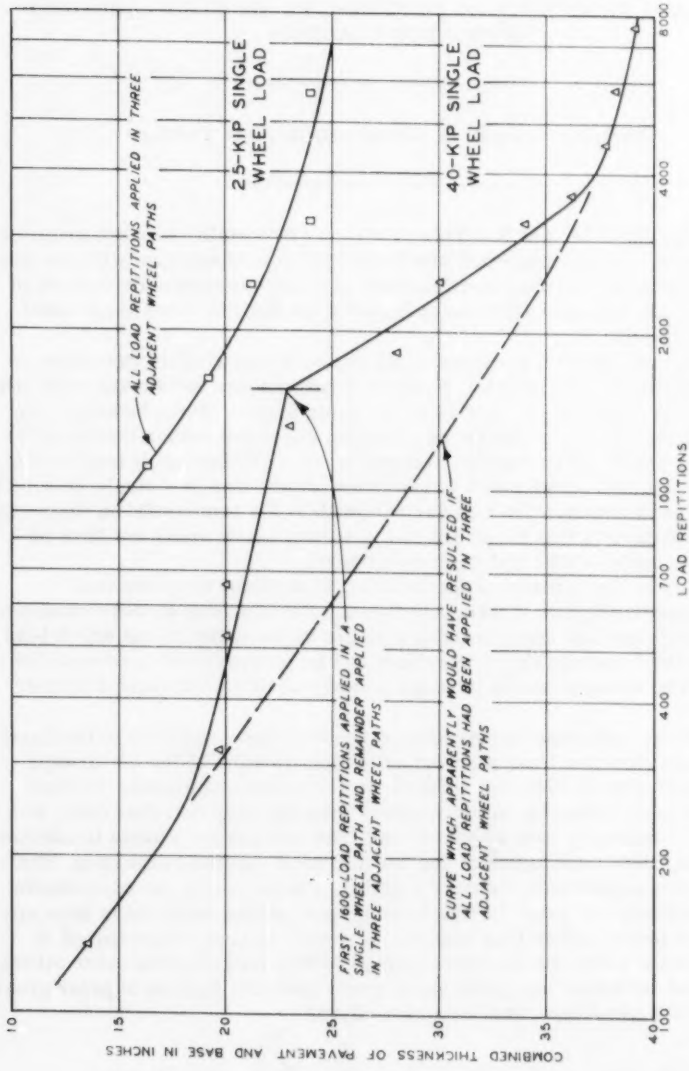

FIG. 1. DEVELOPMENT OF FAILURE IN AN ACCELERATED TRAFFIC TEST SECTION

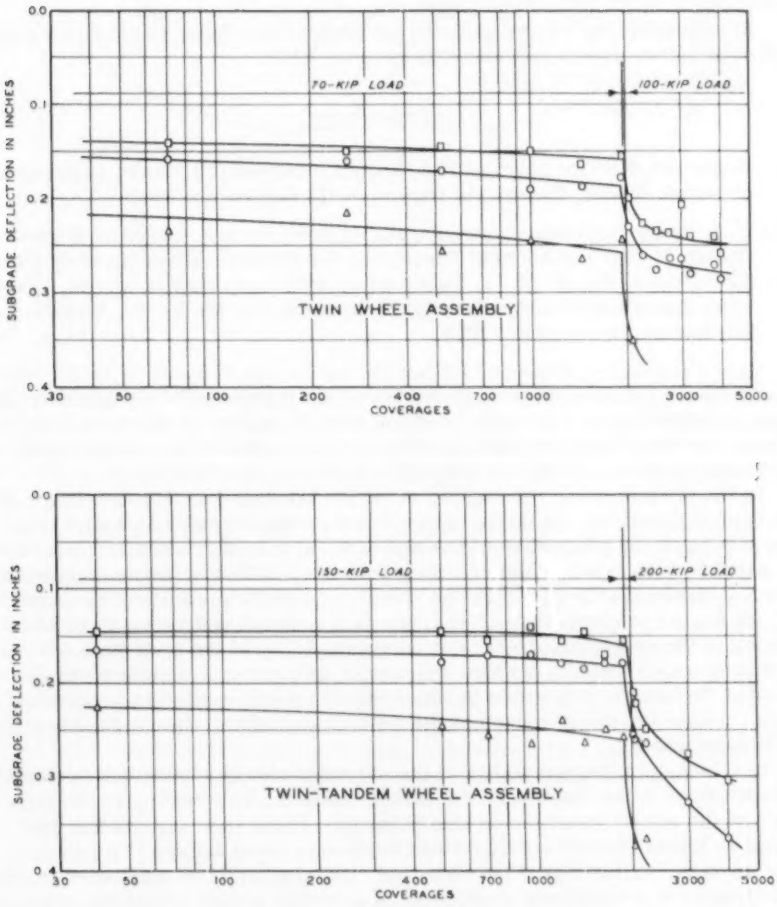

FIG. 2. SUBGRADE DEFLECTION IN AN ACCELERATED TRAFFIC TEST SECTION

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Lastly, the writer wishes to refer to some triaxial testing conducted at the Waterways Experiment Station on a fine, uniform, dry sand in which the specimen was loaded repeatedly. Cycles of loading were 50 or less rather than the thousands of load repetitions reported by the authors. However, just as in the work reported for sands, the results show that successive applications of loading increased the stiffness of the specimen but had no significant effect on its ultimate strength.

In summary, the writer believes the authors are doing an excellent piece of work and wishes to commend them highly on it.

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1. Report on Stockton Runway Test Section, State of California, Department of Public Works, Division of Highways, 28 September, 1942.
2. U. S. Army Engineer Waterways Experiment Station, Corps of Engineers, Design of Flexible Airfield Pavements for Multiple-Wheel Landing Gear Assemblies, Report No. 1, Test Section with Lean Clay Subgrade. Waterways Experiment Station Technical Memorandum No. 3-349, Vicksburg, Mississippi, September, 1952.

D. HUGH TROLLOPE,[1] A. M. ASCE.—This paper marks yet another valuable contribution by Professor Seed and his colleagues on this very important topic. As with the work reported previously one cannot but be impressed with the excellence of the experimental work and its collation.

In the present study two factors in particular emerge. Firstly, that repeated load applications can, under appropriate conditions, affect considerably the stress strain properties of the soil and secondly that intensive research is necessary to evaluate the resulting behavior. It is interesting to note that for the tests described the induced structural effects diminish with increasing initial degree of saturation. It is likely that a significant factor in this behavior is the reduction of effective compressibility of the pore fluid with increasing water content. At high degrees of saturation at least a portion of the applied deviator stress would be absorbed in a build up of pore pressures thus limiting the effectiveness of the contribution of the repeated loading to structural change.

In eliminating the possibility of the observed changes being due to overall densification of the specimens the authors have drawn attention to the influence of the micro structure on this behavior. Could they say whether the samples tested showed a well defined rupture plane at failure? It has occurred to the writer that if the observed stiffening could be attributed to physical changes in a relatively small zone of potential failure within the specimens then this would influence the interpretation of the results to highway performance.

Consider the case of a load repeatedly applied to highway subgrade through a wheel of tyre width 'b'. The inference that could be drawn from these tests is that the zone of failure corresponding to this tyre width would be 'reinforced' by repeated load application (Fig. 1). If now a tire of width b_1 ($> b$) but of the same contact pressure is passed over the subgrade this load would

1. Lecturer, Civ. Eng., Univ. of Melbourne, Victoria, Australia.

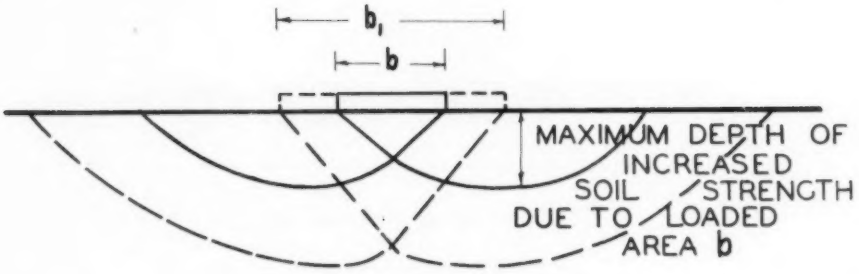

FIGURE 1

create shear stresses in a zone not previously improved by the passing of tyre with width b. This does not invalidate the authors' conclusions concerning the importance of both magnitude and number of repetitions in assessing the effect of repeated loading but rather indicates that under certain conditions the width of the loaded area (tire width) may be an important additional variable.

In conclusion the writer would thank the authors for a most interesting and valuable contribution, and express his appreciation of the work being done at the University of California in this field.

Despite objections by D. P. Krynine[2] it is clear that carefully controlled laboratory investigations of the type under discussion are by far the most important tool in the task of rationalizing the art of pavement design. Systematic field observation can only be expected after the relative influence of the principal variables has been accomplished, and this has certainly not been done in many aspects of soil engineering—particularly pavement engineering. Experience should surely have taught us by this time that skilled laboratory investigation is the most economical way of tackling such problems.

2. Discussion on Paper 1427, J. ASCE, 84:SM2.

GEOLOGICAL FACTORS IN TUNNEL CONSTRUCTION: TOOLS AND TECHNIQUES[a]

Discussion by A. A. Eremin

A. A. EREMIN,[1] A. M. ASCE.—In this paper Mr. Cleaves has justly stressed the importance of geological investigations involved in tunneling.

As early as 1871, while boring the Mont Cenis Railroad Tunnel in the Alps between France and Italy, the longest tunnel in the world at that time, the importance of geological investigations was realized. Later in 1906 during the boring of Simplon Railroad Tunnel, also located in the Alps, a complete section along the tunnel axis with detailed geological information was shown which was included in the economics of tunnel design and construction methods.(1) Today geological investigations are recognized as a basic requirement in economics of tunnel projects.

Mr. Cleaves also shows an interesting summary of modern "tools" used in geological investigation associated with tunneling. As an additional "tool" it is appropriate to add the possibility of consolidating the ground around the tunnel by means of freezing. In boring the Scheld Tunnel at Antwerp, Belgium, freezing prevented a settling of the ground and buildings in the vicinity of the tunnel shaft. Consolidation of the ground around the tunnel bore may also be accomplished by draining the moisture from the soil with the electro-osmosis method.(2)

The author correctly enumerates the advantages of the drill boring method used in the investigation of rock formation. However, by this method it is impossible to measure the moisture content in the soil. Consequently, it is often useful to drill longitudinal bores at the tunnel heading during excavation as they may reveal the concentration of underground water.

Interest in the underground structures and tunnels is increasing, consequently, Mr. Cleaves' paper is timely and warrants careful study.

REFERENCES

1. "Tunnels, Underground Structures, and Air Raid Shelters" by A. A. Eremin, 1958.
2. "Engineering Geology and Geotechnics" by D. P. Krynine and W. R. Judd, 1957.

a. Proc. Paper 1648, May, 1958, by Arthur B. Cleaves.

1. Assoc. Bridge Engr., Bridge Dept. Div. of Highways, State Dept. of Public Works, Sacramento, Calif.

GEOLOGICAL FACTORS IN TUNNEL CONSTRUCTION: GEOLOGY AND TUNNEL DESIGN[a]

Discussion by A. A. Eremin

A. A. EREMIN,[1] A. M. ASCE.—In this paper Mr. Reeves has introduced the question, "What does the tunnel engineer expect the geologist to know?" and reversed, "What the tunnel engineer should know?" In response to the first question proposed by Mr. Reeves' reference to the definition of the engineer-geologist by Prof. Krynine could be made,(1)

> "Engineering Geology differs from geology proper primarily in scope. In the construction of engineering structures depths of over 3000 ft are practically never encountered."

Hence, the work of the tunnel geologist is limited to the upper part of the earth crust.

The importance of the geological investigation of ground in design and construction of tunnels may be illustrated by the incidents which recently occurred while boring major size tunnels.

During construction of the Leadville drain tunnel in Colorado the project manager for the contractor reported completion of 2,126 feet of bore and 30 seconds later the completed length of tunnel was only 1,700 feet. The remaining length of tunnel was filled with mud and gravel which poured through a break in the ground at the tunnel heading.

The second incident which may be cited occurred during construction of the Tecolote Water Tunnel in California. After completing 14,000 feet of bore from the north end of the tunnel the tunnel bore was flooded with 4,000 GPM at a temperature of 118° F. High humidity of about 100% and the existence of methane gas in the bore made the work in the tunnel unusually difficult. At one time even relocation of the tunnel was seriously considered. However, after extensive geological investigation of the ground the work was resumed and the tunnel was successfully completed in its original alignment.

Without any reference to the causes of the incidents it seems obvious that considerable difficulties were avoided by detailed studies of the ground.

In the last century the progress in tunneling has followed the important advancements made in the science of mechanics of stresses in the foundation material.

In 1861 Prof. Rankine developed his well known equation of side pressure on retaining walls. The Rankine equation was successfully applied in design and construction of many recently completed major size tunnels.(2)

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a. Proc. Paper 1649, May, 1958, by A. B. Reeves.

1. Assoc. Bridge Engr., Bridge Dept. Div. of Highways, State Dept. of Public Works, Sacramento, Calif.

In 1880 Prof. Boussinesque developed the well known equation for computation of siresses in the foundation material. The Boussinesque equation is widely used in the analysis of tunnel lining stability.

The recent advancement in methods of geological investigations have resulted in the discovery of various geological factors which impose serious limitations in application of the equations developed by Rankine and by Boussinesque. Therefore, Mr. Reeves' statement made in his conclusion, " . . . by a better appreciation of the professional qualifications of each other, more intelligent results can be obtained" is justified and deserves careful consideration.

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THE STRUCTURE OF COMPACTED CLAYS[a]

Discussions by D. P. Krynine and Dean R. Freitag

D. P. KRYNINE,[1] M. ASCE.—In 1918 or thereabouts, when the writer started his engineering laboratory work on soils, only the old Rankine and Coulomb formulas were at hand. The writer contacted, at that time, the Russian agricultural soil scientists, who were very much advanced in their field, as indeed they are now, and started to work. Because of very unfavorable local conditions at that time the writer was not aware of Terzaghi's work until his "Erdbaumechanik" was published, and soil mechanics was formed. The writer then abandoned the term "Engineering theory of soils," which he had used before in Russia.

Since that time the writer has witnessed two revolutions or, more precisely, two periods of unusually intense activity and creation of new efficient ideas in the province of soil mechanics. The first revolutionary period lasted several years after 1925, when Terzaghi presented his ideas and other engineers, inspired by his work, published numerous papers and discussions. In the subsequent decade the engineering mind was preoccupied mostly with the improvement of sampling and testing, particularly with the triaxial test. Later, wartime conditions did not permit soil engineers to concentrate on the problems of soil behavior, except perhaps the problem of shearing strength of clay, which was arduously studied, but without notable results. A study of the problem of repetitional loadings was also started.

Meanwhile many new ideas were born in adjacent fields. Though soil engineers knew long ago that a portion of particles in actual clay deposits were scale-like, further information on the size and shape of clay particles, and on crystallinity of clays was provided by X-ray analysis and electron microscope. Agricultural soil scientists had had knowledge of the electrical charges in soils for a certain time, and studied with good progress the problems of ion hydration and base exchanges in soils.

Some time ago a group of experts in soil mechanics, physical chemistry, mineralogy, physics, rheology and adjacent fields was organized at M.I.T. by Professor T. W. Lambe, who wrote this paper in consultation with other members of this group. They studied material, analyzed it, and added to it a wealth of their own findings. The present paper is a manifestation of their work. Without making comparisons or drawing parallels, the writer wishes to state his belief that this paper, with the previous works by the same author, is the beginning of another significant revolution in soil mechanics. Whereas at the present time in considering clay we look separately at the clay particles and at the soil water, the new outlook consists in looking into the

a. Proc. Paper 1654, May, 1958, by T. William Lambe.
1. Cons. Engr., Berkeley, Calif.

inseparable team "clay platelets—water," into the orientation of the particles and into the particles themselves.

The following remarks are not intended to oppose the views of the author, but to merely clarify some questionable points or to ask for, or provide some additional information.

Flocculated and Dispersed Clay Structure

To an attentive reader of the paper the difference between the flocculated and dispersed structure is quite clear; but it will be difficult to explain to the average engineer or student that the dispersed structure means smaller pore volume and greater dry density of the soil, as compared with the flocculated one. It seems better to substitute the term "dense" or some other adequate term for "dispersed" structure.

If the faces of a clay particle are charged negatively, there should be both positive and negative charges at the edges. In fact, generally a clay mass is electrically balanced, i.e., contains equal numbers of positive and negative charges distributed throughout the mass. A "structural unit" containing just once all possible combinations of charges and repeating itself according to a certain periodic law may be cut out from the lattices. It is probable that in a cross section of this structural unit and in a portion of this cross section which is the end of the particle, there are both positive and negative charges. Because of a possible anion concentration at the particle ends the total positive charge at the particle end should be weak; and so is the structure thus formed. Isomorphous substitution, probably does not affect this reasoning.

The writer has always been of the opinion that marine clays have larger pores than fresh-water clays. The impression produced on the reader by Fig. 4 is not in accordance with this opinion. If the latter is correct, and if sometime a more permanent publication of this paper is planned, it would be advisable to decrease the whole Fig. 4(b) in order to make the thickness of the particles identical in both Fig. 4(a) and (b).

Diffuse Double Layer

This concept was originally advanced by the famous German physicist Hermann Helmholtz (1821-1894) and subsequently was given various interpretations by different authors. That one given in this paper seems to be the clearest of those the writer knows. The author states that there is adsorbed water (thickness about 10 A) strongly held by the clay particles; (2) the cations are attracted to the clay; (3) water in the double layer is attracted by the cations. Some investigators (e.g., the Russians[1]) state that the boundary of the thin absorbed layer is abrupt, i.e., there is no gradual change in the properties of the absorbed fluid. In any case, it seems that the direct attractive power of the clay particle is exhausted by adsorption and this is the reason why the cations and the water in the double layer outside the adsorbed water layer are attracted by an "induction type of force" as stated in the paper. Thus water is simultaneously attracted to the particles and to the cations, and therefore is in the state of tension. It may be visualized that water threads consisting of water molecules tied to each other by hydrogen bonds thus should be placed in a practically parallel array. Low (ref. 21) describes this water structure as extending upwards of 60 A from the clay surface and becoming less regular, and probably more fluid, with distance from the surface. According to Low, this structure is not identical with that of ice, but both are less dense than "normal" water.

The value of the tensile stress developed in this connection and hence some possible volumetric changes in water depends, of course, on the hydration characteristics of the cations. It appears, therefore, that by using the base exchange process, the water level in a laboratory vessel containing dense clay suspension can be made to fluctuate, though on microscopically small vertical distances. Existence of holes in water is inconceivable. However, in reviewing old literature the writer ran into such terminology.(2)

Some Soil-Water Concepts Interrelated

In connection with the diffuse double layer, the writer believes that the following three soil-water concepts should be in some way interrelated and on some occasions even identical; (1) plastic limit; (2) optimum moisture content; (3) not fully developed diffuse double layer. Obviously, both the plastic limit and the optimum moisture content depend on the hydration characteristic of the cations in the double layer. It seems also that plasticity and thixotropy are related phenomena.

Contact Conditions

When the clay crystals are not in contact, the surface of the adsorbed layer is not necessarily an exact replica of the surface of the crystal; probably it is smooth. When, under pressure, two crystals come in close contact, the two adsorbed layers will form a continuous water envelope in which the two crystals will be suspended, so to speak. If, according to Bolt (ref. 6), a pressure of 100 atms is not sufficient to make two crystals touch each other, clay crystals under engineering structures are never in immediate contact since no such soil pressures as 100 tons per sq. ft. or so exist in practice. This point of view needs criticism and additional study, however.

Slaking Test as a Measure of Structure

The difference in behavior between a dispersed and a flocculated sample of the same clay in a slaking test can be easily explained on the basis of Lambe's theories. The following may be added, however. Kaolinite, halloysite, and many mica clays defy penetration of water and remain intact when submerged, nor does sodium bentonite disintegrate in water.(3) Expandable clays (e.g., montmorillonites) disintegrate in water rapidly.

Electrical Forces and Thixotropy

Inclusion of electrical forces as a component of the clay structure is a very desirable addition to Lambe's theory of clay structure. The writer suspects that the electrical forces appear in clay in the presence of any polar liquid; so far as water is concerned, this is a well known fact. Furthermore, the writer believes that electrical forces that participate in the building of a clay structure, are, at least partially, responsible for the regain in strength (thixotropy) of damaged (e.g., compacted) clay. In this connection the writer wishes to quote a paper printed in Russia.(4)

"Sediments of clay powder are denser in gasoline than in water but at the same time weaker mechanically and exhibit no thixotropy. In water, on the other hand, they do form thixotropic structures." Since the phenomenon of thixotropy is not developed in gasoline and probably in other non-polar liquids

thixotropy is not an intrinsic property of clay. On the other hand, since all compacted clays contain dipolar water, they are all thixotropic, presumably to variable degrees.

CONCLUSION

This paper presents to the reader the results of the fundamental research in the structure of clays. The fundamental research data are ably interwoven with engineering applications, particularly in the field of compaction. In all justice, this is an outstanding engineering paper, and its reading should be recommended to all civil engineers. Particularly this paper is a "must" for soil and foundation engineers if they do not want to lag behind the times in the new soil mechanics era.

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DEAN R. FREITAG,[1] J. M. ASCE.—To the average civil engineer many of the terms and concepts employed in this paper are strange to the ears; and often the full implication of a new concept is difficult to comprehend. In this respect the writer would appreciate a more detailed discussion of the data presented in Fig. 8 of the paper. The author states that at about 200° C all the adsorbed moisture has left the clay and also at that temperature his void ratio data (if the writer interprets the ordinate scale correctly) imply that the soil particles no longer tend to come closer together. In spite of these facts the strength of the soil continues to increase in proportion to the drying temperature. How does this occur?

In several instances Mr. Lambe uses measurements of the shrinkage or swell of compacted soil samples as a means of demonstrating structural differences. In all cases the volume changes are measured relative to the as-compacted volume. Although such measurements graphically portray the state of compaction of the various specimens, the writer wonders if they can be considered a valid indication of structure. It would seem that a more interesting and revealing approach would be to consider the over-all capacity of the soil to undergo volume changes. This approach has been followed by the writer in analyzing the data shown in Fig. A. These data were collected during the course of tests conducted for another purpose and are not as precise

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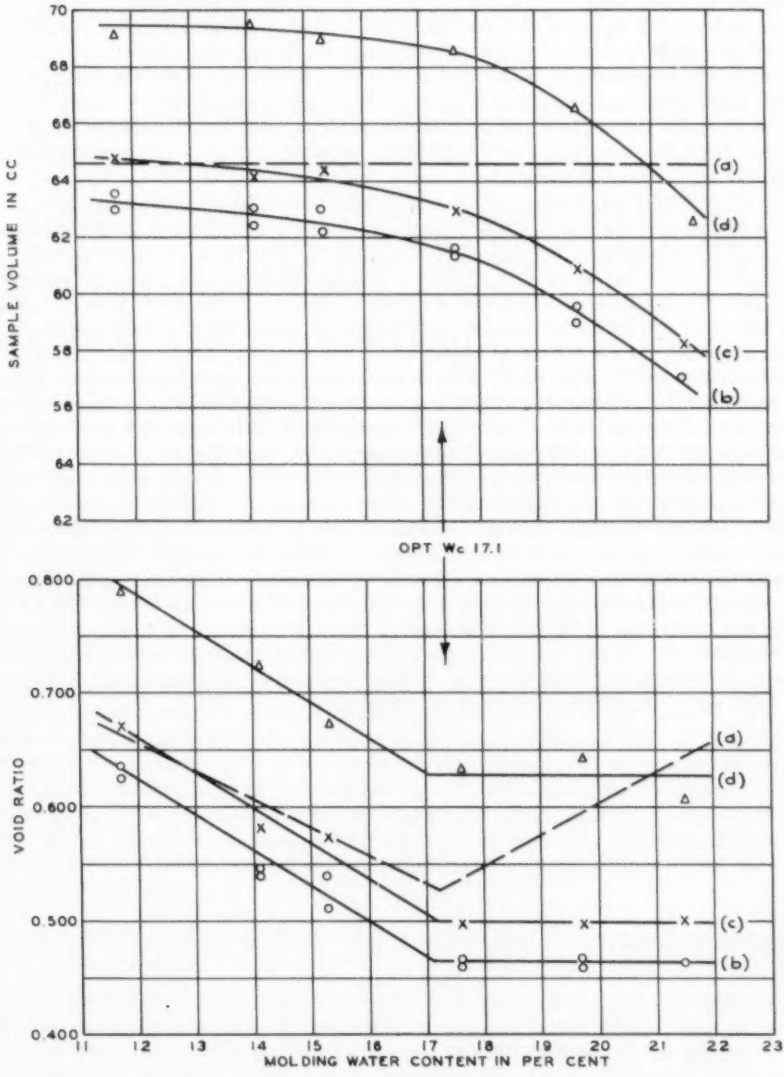

VOLUME-VOID RATIO DATA FOR VICKSBURG LOESS

FIG A

and complete as could be desired but are believed to be adequate to support the discussion.

To obtain these data the various specimens were first dried from the as-compacted state (line a) to equilibrium at 50% relative humidity (line b). Next, half of the specimens were allowed to adsorb water from air at 100% relative humidity (line c) and the remainder were wetted by capillary rise (line d). The specimens represented by line d achieved an average degree of saturation ranging from about 92% for the lowest molding water content to about 88% for the highest molding water content. All specimens had been removed from the molds immediately after molding and were not confined in any way during the test sequence.

It is evident that, based on the as-compacted condition, shrinkage upon drying predominates on the wet side of optimum water content while swelling is most pronounced on the dry side of optimum. There is, however, very little difference in the over-all volume change, particularly when the volume changes are considered in terms of a unit of soil as in the void ratio data plot. If void ratio can be considered an indication, it would appear from these data that, after the initial drying at least, all of the wet side specimens had essentially the same structure. This is not true on the dry side. Thus it should be possible to evaluate the structure of a dry side specimen by a comparison of its void ratio with that of a wet side specimen in a comparable state of moisture equilibrium. However, data from a very few specimens compacted at water contents higher than optimum and using both higher and lower compactive efforts (i.e., a greater and a lesser number of applications of the compacting loading) indicate that a lower void ratio is obtained (at an equilibrium water content) by increasing the compaction effort. Although this is not an unexpected result it implies that a perfectly oriented structure is not obtained merely by compacting wet of optimum. Therefore, it may be necessary to establish a somewhat arbitrary standard of "zero structure" as a base for other measurements. In any event it appears from these data that unless volume change determinations are made at a specified moisture equilibrium they cannot be a very meaningful measure of structure.

THE ENGINEERING BEHAVIOR OF COMPACTED CLAY[a]

Discussion by J. MacNeil Turnbull

J. MacNEIL TURNBULL,[1] A. M. ASCE.—The description and theory of clay engineering behavior given in this paper has considerable interest and useful applications. There are, however, several points on which the conclusions are at variance with those of the writer.

I. C. Effect of Structure on Permeability

Neither of the two soils, Figs. 1 and 2, are true clays. This is indicated by the low optimum moisture content and the high dry density. Since these soils would not contain any appreciable proportions of the true clay minerals they appear to be ill chosen for inclusion as proof of any properties of clays. Because a soil contains more than 30 per cent of particles finer than 0.005 mm. does not imply that it will have any of the properties of a clay. The writer pays more attention to the minimum size of particle in the soil, and the proportion of colloids. Since at any time a colloid content of 30 per cent may consist of silica and be quite useless in a reservoir embankment, hand tests are frequently used to check the results of the mechanical analysis.

II. A. Components of Volume Change

Confirming Experimental Evidence.(3) Is not the noted effect of temperature change largely due to the change in the viscosity of the water? The latter decreases considerably with a temperature rise of 21° C, and would enable the removal of more water from the specimen under a constant load, with a resulting increase in the consolidation. On decreasing the temperature the viscosity of the water increases, and since the actual amount of consolidation for that particular load has been artificially increased by the induced removal of more water than required for equilibrium at a lower temperature, the soil will slowly expand.

II. C. Effect of Molding Water on Compressibility Characteristics of Compacted Clay

What tempering time was given to the soil after the addition of water and mixing before proceeding to make the compaction test? The testing of clays can be quite misleading unless adequate attention is given to the proper tempering of the soil. The writer's practise for clayey soils after addition of water to air-dry soil and thorough mixing, is to leave it to temper in sealed

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a. Proc. Paper 1655, May, 1958, by T. William Lambe.

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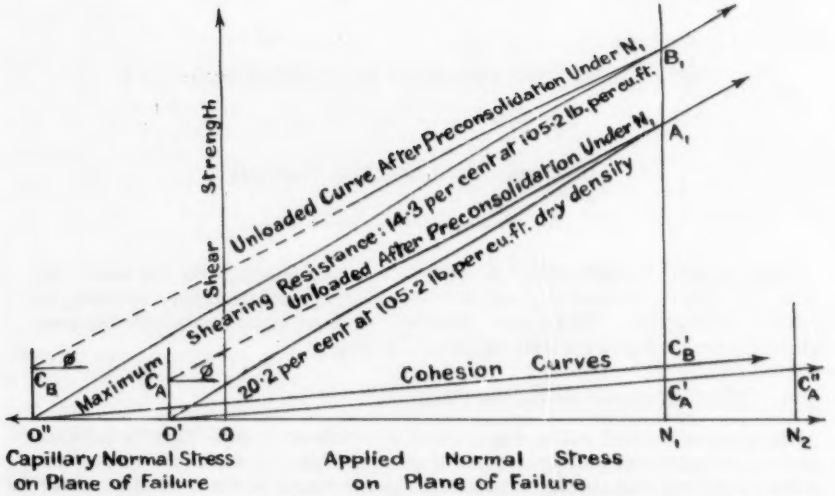

$N_1C'_B = O''C_B$ and $N_1C'_A = O'C_A$, the True Cohesion Under Applied Normal Stress N_1
When Total Normal Stress $O''N_1 = O'N_2$, the True Cohesion $N_1C'_B = N_2C''_A$

FIG.1.- DRAINED TESTS OF UNLIMITED DURATION AT THE SAME DENSITY BUT DIFFERENT MOISTURE CONTENTS

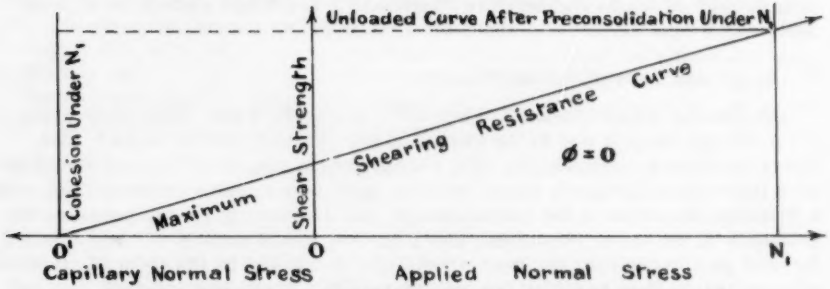

FIG.2 - DRAINED TESTS ON PURELY COHESIVE SOILS

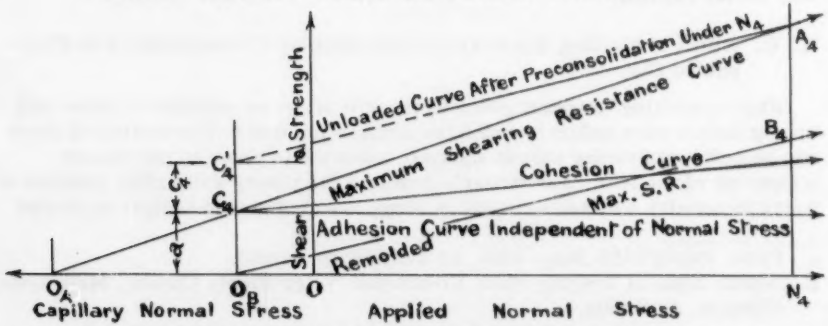

FIG.3.- DRAINED SHEARING RESISTANCE OF ADHESIVE SOILS

jars for at least two weeks before proceeding with compaction and Atterberg Limit tests. At times this has to be extended to three weeks. Unless this is done the clay will not be in its natural state, and a change of 2.5 per cent in the optimum moisture content can easily result. For one soil the optimum moisture content was changed from 30.7 per cent at 88.2 lb. per cu. ft. dry density after tempering for one week, to 28.2 per cent and 91.7 lb. per cu. ft. after three weeks. This was ascribed to the entry of the water into the crystal lattice, providing a more flexible particle with better compaction characteristics, and the removal of water from the surface of the particles where it only hindered compaction. The portion of the curve on the wet leg above 31.5 per cent was common to both tempering periods.

III. C. Cohesion and Friction

The writer is pleased that the author has chosen to avoid the confusion arising from the practice of calling the intercept c "cohesion" and the strength angle $\emptyset$ "internal friction." However, five comments are applicable here:

i) The normal stresses of Figs. 9 and 10 obviously include capillary normal stress (negative pore water pressures), since the normally consolidated curves pass through the origin. The presence of capillary normal stress, which is a real stress, is easily demonstrated by comparison of the strength envelopes of specimens having the same dry density but different moisture contents. The precompressed curves should be straight lines, any departure therefrom being due to errors in the testing technique: such as, too long an interval between the unloading from the preconsolidation stress and the end of the test, or testing the specimen under too small an applied normal stress. Both procedures permit an expansion of the soil with a consequent reduction in strength. Keeping the rate of strain to not less than 0.30 in. per hr., and the applied normal stress to not less than 25 per cent of the precompression stress will reduce this risk. At the same time it is necessary for the effect of these fast rates of strain to be evaluated before the true shearing resistance can be determined. Undisturbed samples, for the same reason, should always be tested under applied normal stresses greater than their preconsolidation stress to determine their true normally consolidated shear strength envelope.

ii) From the appearance of the precompressed curves in Figs. 9 and 10 it would appear that the specimens had been tested very close to the origin. This would not be practicable with clays owing to their large capillary forces. Particularly unfortunate are the curves marked "Engineering Average," which should intercept the normally consolidated curve at the preconsolidation stress. The latter property has often led to the discovery of an erratic test result at the preconsolidation stress.

iii) The statement that "there is no standard test which can identify the contributing fundamental components of clay shear strength" overlooks the fact that this has been done in this laboratory for many years.(1,2) The determination of the true values of all soil characteristics is considered to be essential for the intelligent selection of design values, and to provide a reliable picture of the soil.

iv) Tests to determine the true cohesion and the true internal friction of a compacted clay have demonstrated that, for the same dry density at different moisture contents, the true cohesion is the same at the same

total normal stress (capillary + applied normal stresses) as shown in Fig. 1. The cohesion is independent of the moisture content, which is in line with the fundamental property of true cohesion. At the lower moisture content the free drained and undrained results coincided due to the very small consolidation of the soil.

v) Cohesive clays having no internal friction will, when unloaded from a preconsolidation stress, have a constant shearing resistance equal to the former stress as shown in Fig. 2. A number of such clays have been tested, and present many interesting features. The undrained strength of these clays is not affected by pore pressures like coarser grained soils, and tends to approach the free drained strength.

III. D.

Negative pore pressures (capillary normal stress) exist on the wet side of optimum. In one series of tests on a silty clay loam when compacted at various moisture contents to more than 11.0 per cent on the wet side of an optimum moisture content of 18.9 per cent, the capillary normal stress had a range of 2,500 to 450 lb. per sq. ft. in a free drained test.

The writer considers that the inclusion of capillary normal stress is the only logical manner to complete the interpretation of shearing resistance envelopes. It also provides the only means of obtaining the mean strength of mixtures of different soils.(3)

In conclusion a description of the writer's method of determining the value of the shearing resistance of cemented soils may be of interest. The component due to a chemical or physical bond derived from solutions or heat and pressure, termed "adhesion" can be separated from the true cohesion and internal friction still present in the soil. This type of bond is not affected by the application of an applied normal stress or immersion in water. The soil is tested in the free drained and undrained conditions, and the slope of the "cohesion" curve $A_4C_4^1$ determined (see Fig. 3). Some of the soil is recompacted at the same moisture content and dry density and tested. Since the capillary normal stress is the same for undisturbed and remolded soils, although the strength envelope may be entirely different, the true origin of the "natural soil" O_B is obtained. The intercepts O_BC_4 and $C_4C_4^1$ on the ordinate at O_B supply the values of the adhesion a and true cohesion c_4. The former is a constant under all normal stresses, while the latter will vary with the normal stress, the value c_4 applying to the applied normal stress ON_4 only. In the absence of adhesion the technique has proved of much assistance when testing undisturbed samples where erratic results have been obtained as the origin can then be correctly located.

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PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1859 is identified as 1859 (HY 7) which indicates that the paper is contained in the seventh issue of the Journal of the Hydraulics Division during 1958.

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MAY: 1621(HW2), 1622(HW2), 1623(HW2), 1624(HW2), 1625(HW2), 1626(HW2), 1627(HW2), 1628(HW2), 1629 (ST3), 1630(ST3), 1631(ST3), 1632(ST3), 1633(ST3), 1634(ST3), 1635(ST3), 1636(ST3), 1637(ST3), 1638(ST3), 1639(WW3), 1640(WW3), 1641(WW3), 1642(WW3), 1643(WW3), 1644(WW3), 1645(SM2), 1646(SM2), 1647 (SM2), 1648(SM2), 1649(SM2), 1650(SM2), 1651(HW2), 1652(HW2)[c], 1653(WW3)[c], 1654(SM2), 1655(SM2), 1656(ST3)[c], 1657(SM2)[c].

JUNE: 1658(AT1), 1659(AT1), 1660(HY3), 1661(HY3), 1662(HY3), 1663(HY3), 1664(HY3), 1665(SA3), 1666 (PL2), 1667(PL2), 1668(PL2), 1669(AT1), 1670(PO3), 1671(PO3), 1672(PO3), 1673(PL2), 1674(PL2), 1675 (PO3), 1676(PO3), 1677(SA3), 1678(SA3), 1679(SA3), 1680(SA3), 1681(SA3), 1682(SA3), 1683(PO3), 1684 (HY3), 1685(SA3), 1686(SA3), 1687(PO3), 1688(SA3)[c], 1689(PO3)[c], 1690(HY3)[c], 1691(PL2)[c].

JULY: 1692(EM3), 1693(EM3), 1694(ST4), 1695(ST4), 1696(ST4), 1697(SU2), 1698(SU2), 1699(SU2), 1700(SU2), 1701(SA4), 1702(SA4), 1703(SA4), 1704(SA4), 1705(SA4), 1706(EM3), 1707(ST4), 1708(ST4), 1709(ST4), 1710 (ST4), 1711(ST4), 1712(ST4), 1713(SU2), 1714(SA4), 1715(SA4), 1716(SU2), 1717(SA4), 1718(EM3), 1719 (EM3), 1720(SU2), 1721(ST4)[c], 1722(ST4), 1723(ST4), 1724(EM3)[c].

AUGUST: 1725(HY4), 1726(HY4), 1727(SM3), 1728(SM3), 1729(SM3), 1730(SM3), 1731(SM3), 1732(SM3), 1733 (PO4), 1734(PO4), 1735(PO4), 1736(PO4), 1737(PO4), 1738(PO4), 1739(PO4), 1740(PO4), 1741(PO4), 1742 (PO4), 1743(PO4), 1744(PO4), 1745(PO4), 1746(PO4), 1747(PO4), 1748(PO4), 1749(PO4).

SEPTEMBER: 1750(IR3), 1751(IR3), 1752(IR3), 1753(IR3), 1754(IR3), 1755(ST5), 1756(ST5), 1757(ST5), 1758(ST5), 1759(ST5), 1760(ST5), 1761(ST5), 1762(ST5), 1763(ST5), 1764(ST5), 1765(WW4), 1766(WW4), 1767(WW4), 1768(WW4), 1769(WW4), 1770(WW4), 1771(WW4), 1772(WW4), 1773(WW4), 1774(IR3), 1775 (IR3), 1776(SA5), 1777(SA5), 1778(SA5), 1779(SA5), 1780(SA5), 1781(WW4), 1782(SA5), 1783(SA5), 1784 (IR3)[c], 1785(WW4)[c], 1786(SA5)[c], 1787(ST5)[c], 1788(IR3), 1789(WW4).

OCTOBER: 1790(EM4), 1791(EM4), 1792(EM4), 1793(EM4), 1794(EM4), 1795(HW3), 1796(HW3), 1797(HW3), 1798(HW3), 1799(HW3), 1800(HW3), 1801(HW3), 1802(HW3), 1803(HW3), 1804(HW3), 1805(HW3), 1806 (HY5), 1807(HY5), 1808(HY5), 1809(HY5), 1810(HY5), 1811(HY5), 1812(SM4), 1813(SM4), 1814(ST6), 1815 (ST6), 1816(ST6), 1817(ST6), 1818(ST6), 1819(ST6), 1820(ST6), 1821(ST6), 1822(EM4), 1823(PO5), 1824 (SM4), 1825(SM4), 1826(SM4), 1827(ST6)[c], 1828(SM4)[c], 1829(HW3)[c], 1830(PO5)[c], 1831(EM4)[c], 1832(HY5)[c]

NOVEMBER: 1833(HY6), 1834(HY6), 1835(SA6), 1836(ST7), 1837(ST7), 1838(ST7), 1839(ST7), 1840(ST7), 1841(ST7), 1842(SU3), 1843(SU3), 1844(SU3), 1845(SU3), 1846(SU3), 1847(SA6), 1848(SA6), 1849(SA6), 1850(SA6), 1851(SA6), 1852(SA6), 1853(SA6), 1854(ST7), 1855(SA6)[c], 1856(HY6)[c], 1857(ST7)[c], 1858 (SU3)[c].

DECEMBER: 1859(HY7), 1860(IR4), 1861(IR4), 1862(IR4), 1863(SM5), 1864(SM5), 1865(ST8), 1866(ST8), 1867 (ST8), 1868(PP1), 1869(PP1), 1870(PP1), 1871(PP1), 1872(PP1), 1873(WW5), 1874(WW5), 1875(WW5), 1876 (WW5), 1877(CP2), 1878(ST8), 1879(ST8), 1880(HY7)[c], 1881(SM5)[c], 1882(ST8)[c], 1883(PP1)[c], 1884(WW5)[c], 1885(CP2)[c], 1886(PO6), 1887(PO6), 1888(PO6), 1889(PO6), 1890(HY7), 1891(PP1).

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c. Discussion of several papers, grouped by divisions.

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